Independent Review Team Final Report

East Span San Francisco Oakland Bay Bridge Seismic Safety Project

November 19, 2004

Contract No. 53A0063
Task Order No. 330

In collaboration with

Value Management Strategies, Inc.

San Francisco Oakland Bay Bridge Seismic Retrofit Program

INDEPENDENT REVIEW TEAM - FINAL REPORT

To the reader:

This Final Report reflects the work performed by the Independent Review Team (IRT) for the State of California to document our analysis and findings relating to the East Span of the San Francisco Oakland Bay Bridge (SFOBB) Seismic Safety Retrofit project. This is a very complex project and there are many issues large and small that have been considered in order to advance our study to the point of making final recommendations.

Our recommendations to redesign the main span using a Cable-Stayed bridge are based on broad experience and a sufficient amount of technical analysis provided by the members of the IRT. Ultimately, more engineering work must be performed to complete the project to the point where it is ready for construction. Time is of the essence. There must be a will exercised from all affected parties for the savings anticipated in our report to be realized. With savings forecasted to exceed $600 million and a significant reduction in risk, it is clear that extraordinary efforts will be required on everyone's part in order to best serve the people of the state. We look forward to assisting the State of California and those who will use the SFOBB in advancing the best solution possible for this very important project.

Thomas R. Warne, PE
Chairman, Independent Review Team

Raymond McCabe, PE
C61571

Thomas G. Schmitt, PE

John Lamberson

Tim McGowan

John Hesler

R. Terry Hays

San Francisco Oakland Bay Bridge Seismic Retrofit Program

INDEPENDENT REVIEW TEAM - FINAL REPORT

Table of Contents

Executive Summary

1. EXECUTIVE SUMMARY

1.1 Introduction

This report documents the findings, conclusions, and recommendations by the Independent Review Team (IRT) for the San Francisco Oakland Bay Bridge (SFOBB) Seismic Retrofit Program. The State engaged the IRT on September 3, 2004 to provide an independent analysis of the options, benefits and risk associated with the options to either award the SAS contract, rebid the SAS design or redesign the main span. The IRT is comprised of most of the members of the Independent Review Committee that was formed by the State in September 2003 to recommend actions related to the SAS design at that time. The IRC was supplemented with environmental process experts and additional large bridge construction experts to form an Independent Review Team.

The impetus behind the original formation of the IRC was the single bid on the E2/T1 foundation contract that was 62% over the engineer's estimate. The IRC offered Caltrans a series of recommendations that were combined with a variety of agency-led initiatives, and the project was re-bid. This effort resulted in additional bidders and a re-bid price approximately $50 million lower than the single bid.

In May of 2004, bids were opened on the main span SAS unit after a lengthy bid period, with only a single bid being submitted by a team composed of American Bridge, Nippon Steel, and Fluor. This single bid was for approximately $1.4 billion using foreign steel ($1.8 billion using domestic steel), whereas the engineer's estimate was $780 million. As explained later in the report, a combination of factors contributed to the excessive cost, the first and foremost being the structure type (SAS) and the complexity and the risks associated in building a single tower self-anchored suspension bridge of this magnitude and at this location. This issue resulted in the formation of the IRT to bring together the key members of the IRC to once again assess the viability, risks, and other characteristics of this project. Focus for the IRT was to develop recommendations for the following three available alternatives:

1. Assess the pros and cons for awarding the SAS contract to the American Bridge team
2. Assess the pros and cons of re-bidding the SAS contract with modifications to the contract
3. Assess the pros and cons of redesigning the SAS main span and bidding this alternative

1.2 Initial IRT Findings

In September 2004 the IRT recommended to the State of California that the single bid from American Bridge be rejected for several reasons:

- The state could not legally award the contract without adequate funding in place
- The single bid likely did not reflect the market price for the SAS
- That redesign options existed which could save the state over $500 million and substantially reduce the risks of cost and schedule over-runs likely to occur in building the SAS design

In making the above recommendation, the IRT had also looked into the potential cost savings and schedule impacts associated with several redesign options as described in Section 2.3.

These included:

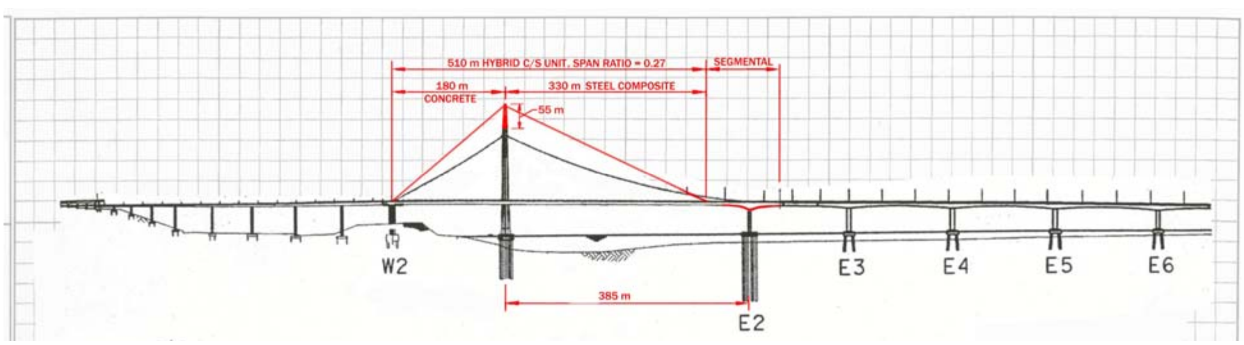
1. Redesign of the SAS to include a concrete tower and a redesigned, simpler superstructure
2. Extension of the Skyway
3. Several cable-stayed options

The preliminary evaluations indicated that:

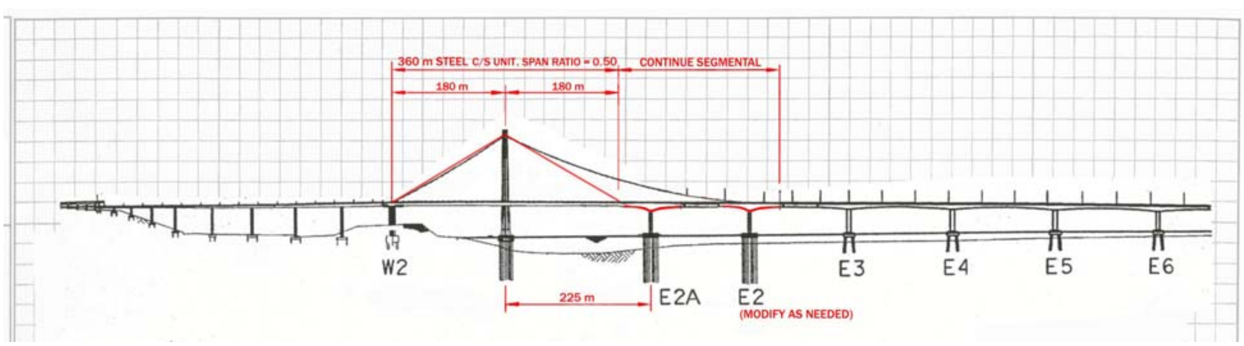
- The savings potential associated with the redesign of the SAS were not of a sufficient magnitude to make this an attractive option.
- The Skyway option would have similar or smaller cost savings than the Cable-Stayed option; it does not represent a "Signature Structure," and was not one of the bridge types recommended by the Metropolitan Transportation Commission (MTC) and Bay Bridge Design Task Force. For these reasons the IRT did not perform further analysis on the Skyway. Basic Skyway information is included in comparison tables, and the IRT developed a construction schedule to satisfy a Caltrans request.
- The cable-stayed options provided the highest level of flexibility, structural efficiency, construction advantages, cost savings, and risk reduction.

Thus the Cable-Stayed option was judged the most attractive. As there are many factors that affect the EIS, technical, schedule, and cost issues differently, three uniquely different cable-stayed concepts were developed, each having certain advantages and disadvantages.

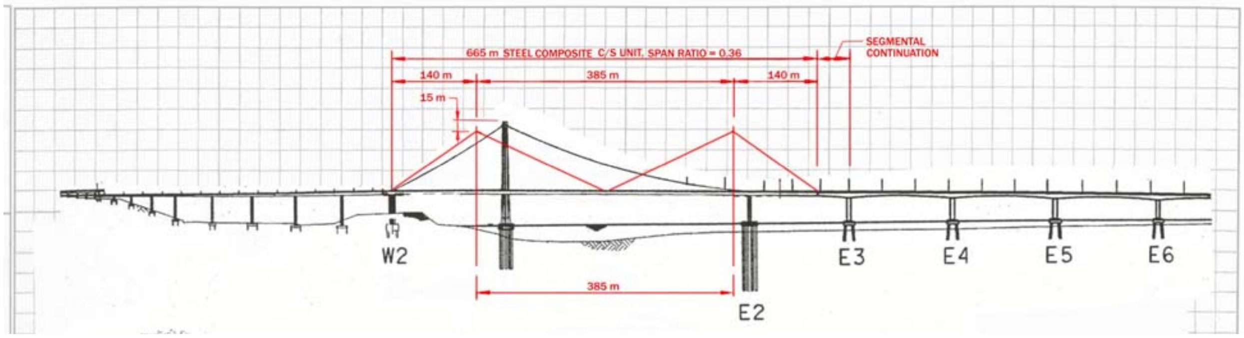
Alternate 1: A single-tower two-span option with 180m – 385m spans



Alternate 2: A single-tower two-span option with 180m – 225m spans



Alternate 3: A two-tower three-span option with 140m – 385m – 140m spans



Alternates 1 and 2 are similar in general appearance to the SAS. While Alternates 1 and 3 provide a navigational span of 385m, Alternate 2 provides only a 225m main span. While Alternate 1 tower height exceeds the 160m limit and Alternate 2 requires an additional pier in the bay, it is our understanding that the requirement for the 385m span and tower height limitations are stakeholder preferences and not design requirements. Discussions on these different redesign choices are given later in Sections 4-6.

1.3 Additional IRT Analysis (Phase 2)

Phase 2 of the IRT's work, which is the focus of this report, consisted of completing a sufficient amount of preliminary technical analysis to further resolve several key issues with respect to the above Cable-Stayed alternatives. The key issues examined in this second phase included:

1. Could the Cable-Stayed alternatives meet the seismic design criteria for the SFOBB
2. Determination of the foundation sizes for the Cable-Stayed alternatives, since this was a major element of the environmental impact with a redesign
3. Assess the environmental consequences of any redesigned bridge options
4. Assess the impacts to YBI and Skyway segments
5. Develop more refined cost estimates and schedule impacts, considering the outcome of items 1 to 3 above

In addition to the preliminary technical analysis, contractor type cost estimates were also developed independently by a Construction Specialist who also provided an independent verification of the construction schedules. An environmental specialist provided independent verification of schedule assumptions related to environmental issues, as well as an assessment of the possible environmental consequences emanating from a redesign. The estimated savings for the Cable-Stayed redesign options include costs of impacts to other contracts, delay costs to the foundation contract, and redesign costs.

The IRT was also required to complete the second phase of the study report by the 19th of November 2004 to facilitate a decision making on the redesign vs. re-bidding of the SAS.

Due to the compressed time schedule and the global nature of the issues to be resolved, the cable-stayed alternatives were prioritized for the second phase investigation in the following manner.

- Alternate 1 was studied first, as this was the one requiring the tallest tower, largest of the foundations, and the highest seismic demands for the towers, foundations, and the interfaces.

- Alternate 3 was studied next, as this was initially estimated to have the shortest construction schedule and the largest of potential cost savings. Also, since it is a two-tower, three-span structural configuration, its technical issues are quite different from the single tower, two-span Alternate 1 or 2.
- The foundation and seismic issues associated with Alternate 2 can be inferred from Alternate 3 due to similar tower height and foundation size. Thus Alternate 2 was set aside initially until the design developments on Alternates 1 and 3 were sufficiently advanced. The limitations on schedule and resources did not permit Alternate 2 to be directly developed. However, the results obtained from Alternates 1 and 3 were sufficient to conclude on the key issues of Alternate 2.

As described later in Section 3, the original SAS foundation/seismic models were used in the preliminary design development process to make a direct comparison with the SAS. As noted later, the analysis procedure adopted is aimed at providing conservative results for this initial study. Further, all design checks for the foundations and interface piers at W2 and E2 were made in accordance with the original design criteria. Design checks for the concrete towers were made with performance criteria more stringent than used for the SAS due to the early stage of development. The seismic performance demands obtained in further stages of design development and analysis are expected to be lower than predicted at this stage. ***This conservative approach provides further confidence in the results of the IRT's analysis***.

1.4 IRT Conclusions

The results of the additional analysis by the IRT of the advantages, issues, and other factors are summarized in Table 1 for easy reference. The major conclusions from the Phase 2 preliminary design development work are:

1. **Seismic Performance:** The Cable-Stayed alternatives can meet or exceed the seismic design criteria for the SFOBB East Span Project. This includes meeting the strain levels with foundation elements, concrete towers, piers, superstructure, shear link performance, and all other elements that govern the seismic performance and safety aspects of the bridge. The concrete towers can be designed to meet the seismic performance requirements of the project. Further information regarding the seismic performance can be found in Sections 3.2.3, 4.2(2), and 6.2(2).
2. **Foundations:** In general, it can be concluded that the foundation sizes and number of piles can remain the same (in some cases the foundations can be smaller) with all of the alternatives. The as-designed SAS foundations can be used for the largest of the Cable-Stayed alternatives (Alternate 1). This assessment is based on similar pile capacity estimates used for the SAS design. However, a review of rock strength data reveals that the pile design used for SAS is extremely conservative. As shown later, the adaptation of a more refined design approach should allow shortening of the drilled shafts at the main tower T1, even for Alternate 1. For other alternates, foundation size can be reduced through redesign, or SAS foundations can be used as is with minor modifications.
3. **Environmental Issues:** The Cable-Stayed design was fully evaluated in the project's Final EIS. Based on the technical analysis performed, the foundation sizes are ***not*** expected to increase for the Cable-Stayed alternatives. There is sufficient reserve capacity in the as-designed SAS foundations at this stage of development that the need to increase their size is hard to comprehend. Further information regarding the foundation capacity can be found in Sections 3.2.3, 4.2(2), and 6.2(2). However, should additional pile capacity be needed for any reason whatsoever, piles can be added within the existing foundation footprints without impacting the foundation sizes.

Thus, the only environmental issues anticipated are the change of structure type from SAS to Cable-Stayed for all three of the alternatives, the height of the tower above elevation 160.0m for Alternate 1, and the need for one additional foundation in the bay for the Alternate 2. The temporary piers required under the SAS design would be eliminated under the Cable-Stayed alternatives.

Both the SAS and cable-stayed designs were fully evaluated as design options under the Preferred Alternative in the SFOBB's Final Environmental Impact Statement (FEIS) that was completed in 2001. The FEIS concluded that the overall environmental impacts of these two options were virtually identical. ***All necessary environmental work can be accomplished through a reevaluation process with minor modifications to existing permits as necessary.*** Additional environmental documentation and modification of existing permits for the Cable-Stayed alternatives can be accomplished in a 9-month period.

Table 2 at the end of the Executive Summary compares the Environmental Intrusions of the various Cabled-Stay alternatives, and the Skyway option to the original SAS design.

4. **Impacts to YBI and Skyway Interfaces:** In general, all of the options considered had little or no impact to the YBI interface. In any case, if some change is needed to the YBI interface, it can be incorporated into the design, as it is still under development. On the Skyway side, some of the schemes (for example, Alternate 1, transition option A) have no impact to the interface, whereas other schemes would have some resolvable design issues. These would simply be designed into the interface and appropriate changes made to the Skyway contract.

5. **Cost Savings:** The estimated net cost savings for Alternates 1 and 3 exceed $600 million. Further, there is an additional estimated savings in excess of $250 million for potential additional costs during construction, as the Cable-Stayed design is judged to have less risk with respect to its fabrication and erection. The same can be inferred for Alternate 2. These cost savings are based on the assumed base price of $1.58 billion ($1.4 billion on the SAS recent bid and $178 million on E2/T1).

6. **Schedule Impacts:** All of the Cable-Stayed alternatives can be constructed by or before the theoretical SAS construction timeline. However, if construction were to proceed on the SAS design, there are overwhelming reasons to expect significant schedule creep during construction; thus, all of the Cable-Stayed alternatives provide significant schedule advantages over the SAS. Detailed schedules were developed for the Cable-Stayed alternates in two scenarios. The first scenario assumed no redesign (except some minor potential adjustments) of the foundations, and the second scenario assumed that the foundations would be significantly redesigned. The detailed schedules developed for the different alternates under these two scenarios are given in Section 7. The feasibility of the use of existing SAS foundations provides schedule advantages in addition to the direct economic advantages.

7. **SAS Risks:** One of the elements of the SAS Bridge that the IRT was asked to review concerned the risk characteristics associated with the construction of the SAS. The single-tower SAS of this size and constructed in this environment is a first-of-a-kind bridge. Even though a bid has been received, there is no reasonable assurance that it could be built within the bid price and schedule. Section 9 details numerous risks associated with constructing the SAS. These risks could add several years to the schedule for completing the SAS design. In addition, it is recommended to budget a construction contingency of $350,000,000 to address these items if the SAS design is

pursued. Experience indicates that first-of-a-kind major bridges have a high potential for construction claims, added costs, and schedule delays.

8. **Project Delivery Method:** There are two primary project delivery methods: Design-Bid-Build and Design-Build. Based on the knowledge and experience of the IRT members, it is recommended that design-build **not** be used for the completion of the Main Span of the SFOBB project if the SAS approach is retained. This is largely due to the complexity of the SAS design and inexperience of Caltrans in utilizing design-build, especially on such a complex project.

   Design-build could be considered with a cable-stayed alternative, as there is not the level of complexity, uncertainty, and inexperience with the cable-stayed design as there is with the SAS. Design-build could be considered for the cable-stayed design if the following conditions were met.

   - Obtain authorization to use design-build from the legislature
   - Validate that the environmental requirements and coordination issues with resource agencies will not be a detriment to the design-build process
   - Prepare Caltrans with the policies and procedures to go forward using design-build
   - Validate that there are costs or time savings associated with using design-build on a cable-stayed alternative

   If the analysis of the project results in affirmative answers to all of these questions, then design-build should be considered. Additionally, it is the recommendation of the IRT that if design-build is utilized for the Cable-Stayed alternative, then Caltrans should immediately secure the services of a project management consultant with experience in the development and management of large design-build projects. The IRT does ***not*** recommend advancing design-build on either the SAS or the Cable-Stayed alternative if the project is going to be self-managed by Caltrans.

1.5 IRT Recommendations

Based on the findings from our study, the IRT recommends proceeding with the redesign of a selected Cable-Stayed alternate. As there are significant cost impacts associated with delays to the current E2/T1 foundation contract, time is of the essence. Alternate 1 offers the most advantages with respect to schedule, and Alternate 3 offers the most in estimated cost savings. Alternate 2 requires evaluation of an additional foundation in the bay, which has potential for schedule delay and offers no real advantage over Alternate 1 or 3.

The IRT offers the following recommendations for the State of California:

1. Immediately adopt the redesign option and select either Cable-Stayed Alternative 1 or 3 as the course of action for moving forward on the main span of the SFOBB.
2. Immediately procure the services of an engineering consulting firm to complete the design work related to the Cable-Stayed option selected in #1 above.
3. Immediately complete a detailed cost analysis for the Cable-Stayed option selected for inclusion in the program budget for the TBSRP for presentation to the legislature.
4. Immediately develop a course of action to deal with the current E2/T1 contract under construction by Kiewit.
5. Immediately start the environmental reevaluation process and any necessary permit modifications.

Table 1: Evaluation of Cable-Stayed Alternatives

		SAS Design	Cable-Stayed Redesign Options			Additional Comments
			Alternate 1	Alternate 2	Alternate 3	
A. Environmental Issues						
1	Tower top elevation	160.0m	217.0m	160.0m	147.0m	Alternate 1 tower height exceeds the 160.0m stipulated for the SAS[1]. Requires a minor revision to the EIS.
2	Navigational span	385.0m	385.0m	225.0m	385.0m	Alternate 2 navigational span is 40% less than the 385.0m for the SAS. Requires a minor revision to the EIS.
3	Structure appearance	Very similar to CS Alternates 1 and 2	Very similar to the SAS	Very similar to the SAS	Somewhat different from the SAS, yet a signature form	Requires a minor revision to the EIS for cable-stayed bridges.
4	Number of foundations	W2, T1 (main tower) and E2	Same as the SAS, with reduced pile lengths at T1	One additional foundation required	Same as the SAS with E2/T1 shifted 40m to the west	Alternate 2 requires a revision to the EIS to allow an additional foundation in the bay.
5	Foundation sizes	Baseline sizes	Same as the SAS	Can be smaller than the SAS	Can be smaller than SAS	No increase in foundation sizes anticipated. For Alternates 2 and 3, the foundation sizes could be reduced[2].
6	Temporary piers in the bay	Required. Significant cost item	Not required	Not required	Not required	Cable-stay superstructures are constructed without temporary piers.
7	Additional NEPA review	None	Reevaluation	Reevaluation	Reevaluation	CS already evaluated in the EIS and was found to have impacts that were virtually identical to that of the SAS.
8	Modification of permits	None	Minor	Moderate	Minor	CS-related changes would be minor. Elimination of temporary piers would be viewed as beneficial by Resources Agencies.

[1] It is our understanding that this is a stakeholder preference
[2] One additional bay foundation is needed for Alternate 2

		SAS Design	Cable-Stayed Redesign Options			Additional Comments
			Alternate 1	Alternate 2	Alternate 3	
B. Seismic Safety & Seismic Performance						
1	Foundations	SAS design criteria – foundations	Same as SAS	Same as SAS	Same as SAS	These elements were checked against the same design criteria as the SAS, using the seismic demands obtained from the same ADINA foundation/seismic model used in the design of the SAS. In the final design, these elements can be designed to be well within the strain limits stipulated.
2	Piers W2 and E2	SAS design criteria – piers	Same as SAS	Same as SAS	Same as SAS	
3	Shear links	SAS design criteria – shear links	Same as SAS	Same as SAS	Same as SAS	
4	Superstructure	SAS design criteria – super structure	Same as SAS	Same as SAS	Same as SAS	
5	Concrete tower	Essentially elastic response under SEE	Meets or exceeds SAS performance criteria	Meets or exceeds SAS performance design criteria	Meets or exceeds SAS performance design criteria	The strain limits used to check the seismic performance of the concrete tower for SEE are the same as those used for FEE in the SAS design[3]. In addition, the cable arrangement provides considerably more global stability and enhances overall seismic performance and safety.
6	Overall seismic safety	Essentially elastic response under SEE	Meets or exceeds SAS performance criteria	Meets or exceeds SAS performance design criteria	Meets or exceeds SAS performance design criteria	
C. Interface Issues						
1	YBI side	Baseline case	Not an Issue	Not an Issue	Not an Issue	The YBI side is still in the design phase. Any modifications needed are expected to be relatively minor and can be incorporated into the design
2	Skyway side	Baseline case	Transition Option A has no impact Transition Option B require some design evaluation	Transition Option A has no impact Transition Option B require some design evaluation	Requires design revision to shorten the length of the Skyway superstructure. Relatively minor change to the design	All three CS alternates can be used in a manner that requires little or no change to the Skyway. However, as with Alternate 1, Transition Option B, there are benefits to be gained if some changes can be made to the Skyway.

[3] The SAS design criteria allow some limited damage at the higher magnitude SEE event and allows no damage at the lower magnitude FEE event. The concrete tower design checks under the SEE event meets the no-damage requirements stipulated for the FEE event.

		SAS Design	Cable-Stayed Redesign Options			Additional Comments
			Alternate 1	Alternate 2	Alternate 3	
D. Other						
1	Design Life	150 years (for the SAS and Skyway). Baseline design life	Same as the SAS and Skyway	Same as the SAS and Skyway	Same as the SAS and Skyway	The deck design and performance for the cable-stayed options would be the same as the SAS or Skyway, depending on the final deck type selection[4].
E. Schedule						
1	No foundation redesign	—	Completion in late 2010	Not applicable	Completion in early 2010[5]	There is some schedule advantage with Alternate 1, as the existing foundations can be used as-is (with only minor modifications)[6].
2	Foundations redesigned	—	Not applicable	Completion in late 2010	Completion in late 2010	
F. Cost Savings						
1	Savings in construction	—	$673,000,000	$700,000,000[7]	$829,000,000	The additional savings is the estimated difference between the potential for construction cost additions between the SAS and the CS.
2	Additional savings	—	$250,000,000	$250,000,000	$250,000,000	
3	Total potential savings	—	$923,000,000	$950,000,000	$1,079,000,000	

[4] See Section 3.2.1

[5] The existing foundations are too big for optimal design of this alternate. Redesign is preferred from a technical point of view to achieve better overall performance

[6] The potential foundation contract delay claims can best be minimized with Alternate 1

[7] Would depend on the terms and conditions of modifications to the existing foundation contract to include the additional foundation or potential re-bidding of the foundation contract

Table 2: Environmental Intrusion Comparison

	SAS	Skyway	Cable Stayed Options		
			Alternate 1	Alternate 2	Alternate 3
Maximum Tower Height	160 meters	None	217 meters	160 meters	147 meters
Cable System Appearance	Sag cable with vertical taut cables	None	Inclined taut cables	Inclined taut cables	Inclined taut cables
Visual Impact of Main Span [a]	48,310 m^2 Baseline Signature Span	8,500 m^2 (C) 5,700 m^2(S) No Signature Span	57,885 m^2 Enhanced Signature Span	30,600 m^2 Reduced Size of Signature Span	52,200 m^2 Enhanced Signature Span
Total Piers in Bay	44 [b]	45	44	45	44
Net Fill in Bay (Acres)	2.61 [c]	2.66	2.61	2.60	2.60
Temporary Foundations in Bay	Yes	Concrete – No Steel – Yes	No	No	No
Deck Height at Highest Point	Baseline	Same	Same	Same	Same
Superstructure Profile Thickness	5.5 meters	15 meters (C) 10 meters (S)	5.0 meters	5.0 meters	5.0 meters
Navigational Channel (Clearance)	42.6 meters	33.1 meters (C) [d] 38.1 meters (S)	43.1 meters	43.1 meters	43.1 meters
Navigational Channel (Width)	385 meters	260 meters (C) 205 meters (S)	385 meters	225 meters	385 meters
Biological Impact	Baseline	Slight Increase	No change	Slight reduction	Slight reduction
Historic/Cultural Resources	Baseline	No change	No change	No change	No change
Archeological Impacts	Baseline	No change	No change	No change	No change

[a] Visual Impact of Main Span considers the total square meters for the tower, cables and deck in the elevation view. The tower below the deck is not included in the calculations
[b] Source – Figure 2-10.1 of Final EIS
[c] Source – Table 4.9-2 of Final EIS
[d] 22% reduction from the minimum clearances shown for the SAS

Introduction

2. INTRODUCTION

2.1 Introduction

This Final Report documents the findings and conclusions of the Independent Review Team (IRT) for the San Francisco Oakland Bay Bridge (SFOBB) Seismic Retrofit Program. It covers the work of the IRT from September 7 through November 19, 2004. This report contains an overview of the current status of the SFOBB, an analysis of alternatives available as well as conclusions and recommendations to the State of California for advancing this project to completion.

2.2 Background

The Toll Bridge Seismic Retrofit Program (TBSRP) was established in response to the need to preserve critical structures in the state against possible future seismic events. The program is composed of a number of projects the most significant of which is the replacement of the East Span of the San Francisco Oakland Bay Bridge (SFOBB). This is the last major project to be completed as part of the TBSRP. The East Span replacement is divided into 16 contracts, the most notable of which is the signature main span located just east of Yerba Buena Island known as the Self Anchored Suspension (SAS) bridge.

The SAS bridge was selected in 1998 through an extensive public process and adopted as the preferred alternative for the Environmental Impact Statement (EIS) and the Record of Decision (ROD) signed by the Federal Highway Administration (FHWA).

In May of 2004, bids were opened on the SAS with only a single bid being submitted by a team composed of American Bridge, Nippon Steel and Flour. This single bid was for approximately $1.4 billion (foreign steel bid) and was significantly over the engineer's estimate for the work of $780 million.

The Independent Review Team (IRT) was first constituted for the San Francisco Oakland Bay Bridge Seismic Safety Retrofit Program on September 7, 2004. Thomas R. Warne, PE, a nationally recognized transportation professional, was invited to chair the effort and additional individuals from the transportation industry were invited to complete the membership of the IRT. Each member of the IRT is a professional with specific expertise in some area of large project delivery or other such elements relative to the TBSRP. Abbreviated curricula vitae for each member of the IRT are found in Appendix A. The impetus behind the original formation of the IRT was the single bid on the SAS foundation contract that was almost 80% over the engineer's estimate.

In September 2004, the IRT was asked to offer recommendations to the State and Caltrans regarding the disposition of the single bid received on the SAS (Superstructure and Tower) Contract in May. In its Executive Summary dated September 30, 2004 the IRT recommended to the State of California that the single bid from American Bridge be rejected for several reasons:

- The state had insufficient funding to award the bid and could not legally do so
- The single bid likely did not reflect the market price for the SAS
- That redesign options existed, including a cable stayed alternative which could possibly save the state over $500 million

Subsequently, the state rejected the single bid and launched a new process designed to bring to conclusion the most responsible decisions relating to the completion of the East Span of the SFOBB. The work performed by the IRT since its inception in September 2004 is documented in this Final Report and is based upon the scope of work detailed in the next section of this document.

Scope of Work

The scope of work for the IRT is divided into two phases. This first phase reflects the work completed by the Independent Review Team after it was activated on September 3, 2004 but prior to the September 30th decision to reject the single SAS bid. Here the express purpose was to offer input and recommendations regarding alternatives for the State of California in advancing the SAS main span project and the appropriate action relating to the single bid received from the team composed of American Bridge, Nippon Steel and Fluor.

In doing this, the IRT was asked to assess the viability, risks and other characteristics of the following three options for moving ahead with the Main Span project.

Option 1 - Award the contract to the American Bridge team
Option 2 - Rebid the SAS contract with modified terms and conditions
Option 3 - Redesign the main span

The work of the IRT would include an assessment of the pros and cons for advancing each of these options so that the state could determine the comparative advantages and disadvantages of each.

The second phase of the IRT's work consisted of performing sufficient technical analysis of Option 3 with the assumption of a possible cable stayed approach. This work would; determine if the cable stayed options could meet seismic criteria for the SFOBB Project, determine what, if any, modifications were necessary to the foundations currently planned and/or under construction and assess the environmental consequences of any redesigned bridge options. In addition, appropriate analysis and cost impacts for the main span project as well as adjacent projects were to be determined. This report will offer the results of the IRT's work in both phases of this project.

Phase 1: Three Options-September 30, 2004

The IRT reviewed the single bid condition for the SAS and was tasked by the State of California to offer alternative courses of action. Ultimately, the IRT concluded that there were three available options to the state for advancing the main span work of the East Span of the SFOBB. They were:

Option 1-Award the contract to the American Bridge team
Option 2-Rebid the SAS contract with modified terms and conditions
Option 3-Redesign the main span

Each of these options has pros and cons, as well as certain elements of risk. A brief summary of the pros and cons for each option including some commentary is provided below:

Option 1-Award the contract to the American Bridge team

Pros

1. Caltrans has a bid in hand
2. Known starting or base price for the work
3. No further environmental analysis or permitting is required

4. The project continues to advance towards completion

Cons

1. Single bid doesn't ensure the most competitive price for the state
2. Significant constructibility concerns expressed by contractors
3. Complex fabrication issues with bridge components
4. One-of-a-kind bridge with little or no US experience in its construction
5. High risk of schedule delays
6. High risk of cost overruns
7. Limited sources for some specialty materials
8. High cost ($200-300 million) for temporary throw-away work

This first option called for extending the current period for contract award to the American Bridge team for an additional term of five months or more so that sufficient funding could be secured to finalize this contract. The timing of this option was full of uncertainties and the outcome of the final contract even more so. Under current procurement code in California, the state is unable to commit to any price adjustment or other concessions with a contractor prior to entering into a contract with that organization.

Therefore, American Bridge would be required to hold their price constant from May 2004 until the state was in a position to execute a contract with them. With inflation in construction in the range of 5% per year and some materials, such as steel and cement, changing even more, it would be unfair for the state to expect American Bridge to hold their prices firm under such circumstances for any long period of time.

Of equal importance is the fact that the state only received one bid tender. The IRT accepts that American Bridge has stated this to be a fair price for the work to be performed. However, without the opportunity for competition there is little the state can rely on about this price relative to the true value for this work if priced in a competitive environment. It is generally accepted in the contracting industry that owners achieve the most cost effective price when at least two bidders compete. When multiple bidders compete owners then know the market price of their project. At this point, Caltrans does not have this crucial information.

Perhaps most significant is the fact that the state does not have sufficient funding to award the contract and is legally prohibited from doing so. Thus, awarding the SAS to the single bidder wasn't a viable option on September 30, 2004 even if this was a desirable course.

Option 2-Rebid the SAS with contract modifications

Pros

1. Possibility of one more bidder creates some measure of competition and potentially reduces project costs
2. No further environmental analysis or permitting is required

Cons

1. Some project delay due to the timeframe required to rebid the project
2. Significant constructibility concerns expressed by contractors
3. Complex fabrication issues with bridge components
4. One-of-a-kind bridge with little or no US experience in its construction
5. High risk of schedule delays
6. High risk of cost overruns
7. Limited sources for some specialty materials
8. High cost ($200-300 million) for temporary throw away work

This option has many of the same pros and cons as Option 1-Award the contract to the American Bridge team. Two significant differences lie in the fact that Caltrans can modify the contract terms and conditions in order to create a better bidding environment and the hope that additional contractor teams will want to compete for the SAS work. In the first case, contract terms and conditions can make a substantial difference in how contractors view a project and ultimately price the work. If owners are fair about risk allocation, offer clear terms and conditions which reflect the complexity of the work and otherwise create a favorable environment for pursuit of the construction activities, this encourages contractors to offer competitive prices. This can be done while still guarding the public trust.

Regarding the second point, it is anticipated that at least two teams would need to offer bids on the SAS rebid to achieve some measure of competition. More would be desirable but given the limited population of contractors/contractor teams capable of building a project like the SAS little likelihood exists that the competition would include more than two teams. The risk to the state in following this option would occur if no team chooses to bid the SAS the second time around or if only one team bids it again. It is the opinion of the IRT that California would then have little choice but to award the SAS on a rebid regardless of the prices submitted on the second round of bidding. The history of failing to award contracts on the SFOBB will begin to work against Caltrans given the rejection of the original E2/T1 bids in 2003 and now the SAS bids in 2004. The contracting community expends considerable sums and good will in bidding state work and the process of bidding and rebidding work is damaging to the reputation of the state and will ultimately result in higher overall prices from the industry.

Option 3-Redesign the main span

Pros

1. Potential for significant cost savings to the state
2. Ability to meet the schedule objectives of the project and complete the work by 2011
3. Increased competition
4. Ability to build a "signature" type structure
5. Availability of materials
6. Fabrication of materials is simplified

Cons

1. Possible conflicts with the E2/T1 SAS foundation contract
2. Cost of redesign of the main span
3. Cost of interface changes with the Skyway and Transition contracts
4. Additional environmental/permitting work
5. Time to complete additional environmental/permitting work
6. Need to change legislation regarding the SAS design
7. Possible schedule impacts to other projects

The final option available to the State of California is the redesign of the main span and the construction of an alternative bridge type. Essentially, this option recognizes that alternative bridge types could be constructed which still achieve project objectives. These include modifications to the SAS design, the extension of the current skyway bridge over the main span and the use of a cable-stayed design. The cost savings to the state are substantial for the latter two alternative bridge types when compared to the expense of building the SAS as currently envisioned. After considerable analysis by the IRT the cable-stayed alternative was ultimately considered to be most desirable for replacing the SAS design. This analysis will be presented later in this report.

In order for there to be an appreciation for the IRT's conclusion that a cable stayed bridge is most advantageous to the state a brief review of all three redesign options will be provided.

The as bid price for the Self Anchored Suspension Bridge (Superstructure, Towers and Foundations E2, T1) is approximately $1.6 billion with the foreign steel bid. This total includes both the cost for the SAS Superstructure and Tower as well as the E2/T1 foundation contract. It equates to a cost per square foot of deck area of over $4000 which is significantly out of the cost range of more typical (cable stayed bridges) of the same span length. While the project's seismic criteria and local construction conditions can account for some of this difference, the following factors among others also contributed largely to the high cost:

1. Uniqueness of SAS design
2. Construction risk
3. Lack of competition
4. Steel fabrication complexity
5. Construction requirements (Need for Temporary Piers)

2.3 Main Span Redesign Options

Based on experience and the significant amount of engineering work performed on the East Span project to date, the following redesign alternatives are expected to result in cost savings of various amounts:

1. Redesign SAS
2. Continue Skyway
3. Redesign as Cable Stayed

Each redesign option is briefly reviewed in the following:

2.3.1 Redesign the SAS

The current Self Anchored Suspension bridge could be redesigned by changing the steel tower to a concrete tower and the steel orthotropic superstructure to a steel composite (lightweight concrete) superstructure.

Advantages:

- Reduced expensive steel fabrication
- Concrete construction familiar to local construction community
- Potential for increased competition
- More adaptable to temporary stayed construction to avoid costly temporary piers
- Can meet project schedule if environmental time frame is achieved

Disadvantages:

- Larger foundations (Environmental Issue)
- Larger Suspension Cable

Potential Savings: $100 – 200 million

2.3.2 Continue Skyway

The current skyway design (box girder type bridge) could be continued to Pier W2 with various design modifications. A concrete or steel box girder superstructure could be used. This design would require an additional costly foundation in the bay.

Advantages:

- Continuation of a bridge design and associated construction methodology more familiar to US Contractors
- Less risk for cost and schedule overruns.
- Potential for more competition
- Can Meet Project Schedule if Environmental Time Frame is Achieved (See Environmental Discussion)

Disadvantages:

- Not a Signature Bridge Solution
- Additional Costly Foundation in Bay
- Potential for Single Bidder for concrete box girder (Advantage to Current Skyway Contractor)
- Higher Degree of Environmental Impact due to Additional Pier Requirements

Potential Savings: Greater than $500 million[8]

Additional Discussion on the Skyway Option: Extending the skyway by using a box girder structure for the main span between hinge A and K is a viable option for the redesign. For this option, both concrete and steel superstructures are possible and are further described below.

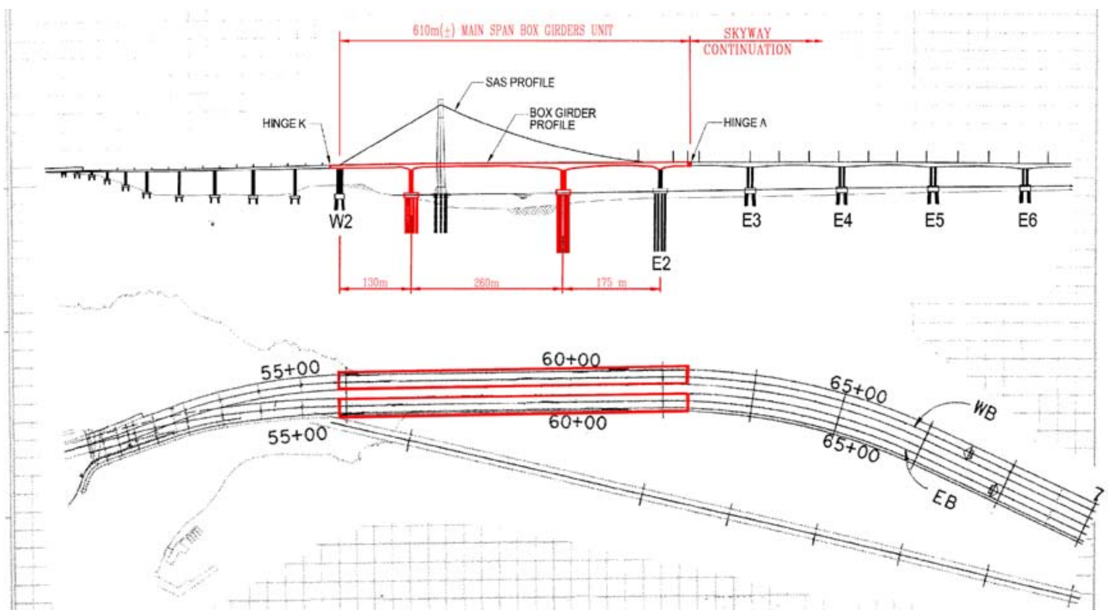


Figure 2.1: Skyway Concrete Box Girder Re-Design Option

1) Concrete Box Girder - In order to layout the span arrangement for the concrete box girder, the construction methodology is important. With a main span greater than 200 meters, the optimum construction method is balanced cantilever using cast in place construction (with form travelers) or a combination of cast in place and precast construction (since skyway casting yard is already set up). In order to reach hinge A,

[8] Preliminary estimate based on the cost of Skyway. The increased span lengths and deeper box girders required for the main span was not factored in to this preliminary estimate

cantilevers of 40 meters each side of pier E2 are required. This leaves 520 meters to reach pier W2. Using balanced cantilever construction from piers T1 and E2A results in a three span arrangement of 130m-260m-175m (see Fig. 2.1). For this arrangement pier T1 is shifted 50 meters to the west. The 260 meter span would be the longest span for this bridge type in the US (however only 14 percent greater than the Houston Ship Channel Bridge which has a main span of 228 meters) thus constructability should not be a problem. This option was not recommended for further study for the following reasons:

a. Our experience indicates that this solution would be more expensive than Cable Stay Alternate 2
b. This bridge has significantly more mass than CS Alt. 2 and thus greater foundation impacts
c. Requires additional pier (Pier E2A) in the bay
d. Bridge type not considered a signature bridge and not a structure type originally adopted by the MTC and Bay Bridge Design Task Force

2) Steel Box Girder - The span arrangement for the steel girder option is not as construction dependent as the concrete solution. Keeping piers W2,T1 and E2 in their current location would result in a three span arrangement of 180m-205m-180m with a 40m section cantilevering beyond pier E2 to hinge A. While the end spans are longer than optimum it was felt to be more desirable to keep the piers in their current location if possible. Additional pier E2A would be positioned 180 meters west of pier E2. A steel orthotropic deck was assumed for this option. Construction methodology for this bridge would be to construct the end spans using temporary supports, cantilever into the main span from piers T1 and E2A a certain distance and then lift the central girder section from the completed cantilevers ends. Large steel box girder sections (approx. 35 feet deep at the piers) would be fabricated, barged to the site, and lifted onto the piers and temporary supports. The center section would be fabricated full length, barged to the site and lifted using a jacking arrangement from the completed cantilevers. This option was not recommended for further study for the following reasons:

    1) Our experience indicates that this option would be more expensive than Cable Stay Alternate 2
    2) Fabrication of the large steel orthotropic sections is costly and not possible for US fabricators without significant up front set up costs.
    3) Requires additional pier (Pier E2A) in the bay.
    4) Require expensive temporary supports(similar to the SAS) in the deep portions of the bay and on the island
    5) Not considered a signature structure and not a structure type originally adopted by the MTC and Bay Bridge Design Task Force.

2.3.3 Redesign to a Cable Stayed Bridge

Cable Stayed Bridges have continued to gain world wide acceptance due to their beauty and economy. The cable stayed bridge was one of the alternates studied during the Type Selection Phase of the project in 1998 and gained stakeholder and public acceptance. Their advantages and cost efficiency are primarily related to the following factors:

1) Improved constructability

- Proven and faster superstructure construction
- Temporary piers not necessary in the Bay
- Contractor familiarity with their construction
- Simpler structural elements and details

2) High structural efficiency

- Traditional superstructure construction (steel composite) familiar to industry
- Concrete Towers
- U.S. Stay cable technology

3) Predictable costs above foundation level

4) Greatly increased competition

- Reduced construction risk over SAS
- General contractor pool – US Cable stayed bridges generally attract 4 or more bidders
- Steel framing familiar to US steel fabricators
- Multiple cable suppliers

The following cable stay redesign options are feasible given the current constraints in the project: Each is based on the use of concrete towers (single tower between roadways), a steel composite lightweight concrete superstructure, and two planes of cable stays (similar to the preferred arrangement studied in Type Selection Phase in 1998).

1) 180 m – 385 m Two Span (Figure 2.2)

- Single Tower (55 m taller than SAS)
- Moderate change in visual form
- Same foundation locations as current SAS
- Possible larger T1 foundation
- Can meet project schedule provided environmental schedule can be achieved (See Environmental Discussion)

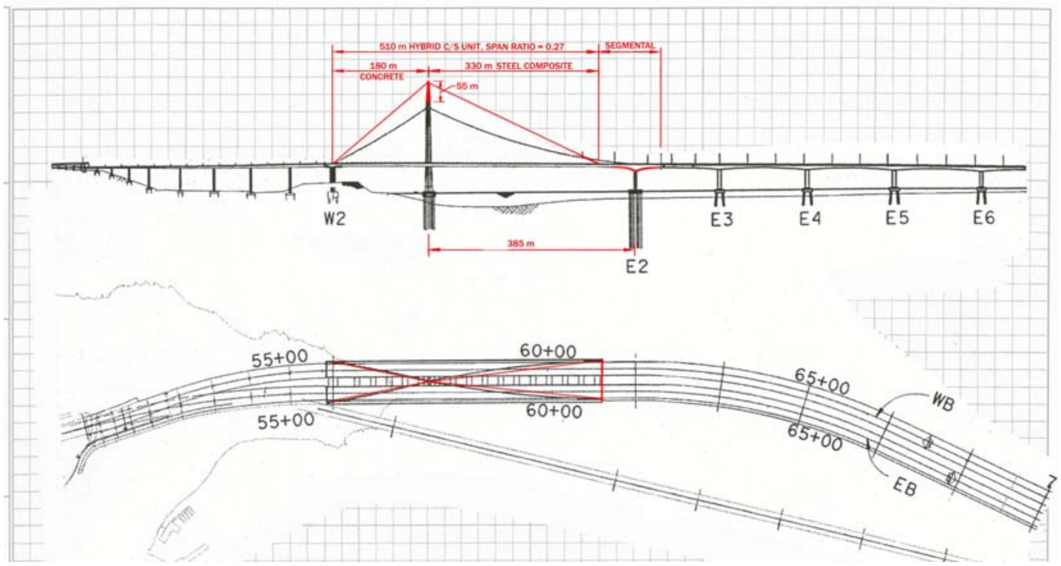


Figure 2.2: Cable-Stayed Alternate 1

2) 180 m – 225 m Two Span (Figure 2.3)

- Single Tower (same height as SAS)
- Moderate change in visual form
- Same foundation locations as current SAS
- One additional pier required in Bay
- Possible same size E2, T1 foundations
- Can meet project schedule provided environmental schedule can be achieved (See Environmental Discussion)

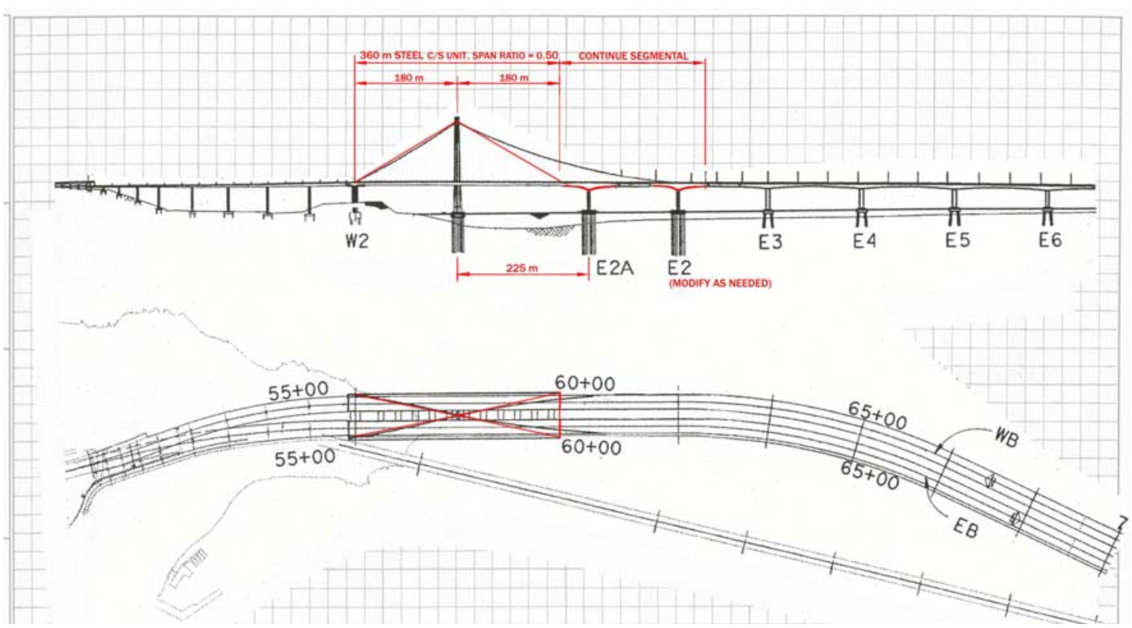


Figure 2.3: Cable-Stayed Alternate 2

3) 140 m – 385 m – 140 m Three Span (Figure 2.4)

- Two towers (same height as SAS)
- More extensive change in visual form

- Same number of foundations as SAS however require moderate shift in location
- Possible same size E2, T1 foundations
- Can meet/shorten project schedule provided environmental schedule can be achieved (See Environmental Discussion)

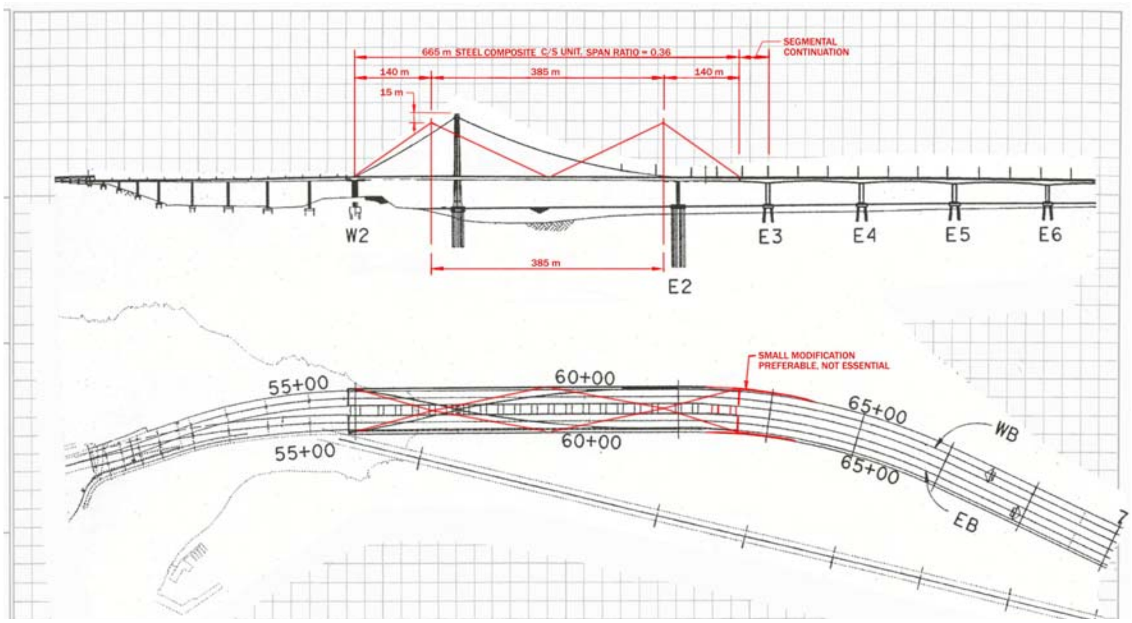


Figure 2.4: Cable-Stayed Alternate 3

2.4 Relevant Previous Work - Cable Stayed Alternatives

As noted below, there have been some previous studies of a cable-stayed alternative for this location, and many of the performance aspects have been investigated and verified that such a design can meet the same design standards as the SAS

In early 1998, the TY Lin/Moffatt Nichol Joint Venture performed a 30% design for a cable stayed main span segment of the SFOBB east spans. The 30% design level was to incorporate seismic related requirements into the cost estimates. A special focus was placed upon foundations, piers, structural configuration and fuses. These are similar areas that the IRT is focusing on in our analysis of the cable stay options. The bridge consisted of two spans (215m, 275m), utilized a concrete tower with shear links and a steel composite superstructure with lightweight concrete deck. An alternate deck system using steel orthotropic deck was also included. Alternate 1 of the IRT cable stay option is very similar to the bridge studied by the Joint Venture except the spans are 180m, 385m representing an increase in total length of only 15%. The results of the analysis and design are summarized in a report titled "SFOBB East Span, Seismic Safety Project, 30% Type Selection, May 1998".

Some of the major conclusions of that document are the following:

1) Concrete Tower with Shear Links – "The composite design of this tower section combines the economy of reinforced concrete tower construction with the exceptional ductility of compact steel links, using both materials to their greatest advantage. The resulting system is a great improvement over either all concrete or all steel systems in terms of value, performance, and maintainability.

2) Wind Design – Cable stayed single tower bridge is expected to be extremely stable in both horizontal and vertical modes of vibration. The deck section (with the bicycle path on the windward or leeward side) became progressively more and more stable as the wind speed increased up to a full scale equivalent wind speed of over 225 m/s. No critical flutter velocity was detected.

3) Seismic Performance – The seismic performance rankings of the single tower cable stayed bridge was a 9.5 out of possible 10. A similar ranking was given for the single tower self anchored suspension bridge. This ranking indicates that the cable stayed bridge would perform exceptionally well in a seismic event and meet the seismic criteria for the project.

A paper titled "New Developments in Cable Stayed Bridge Design, San Francisco" by David Goodyear and John Sun (both of TY Lin) (Appendix B) further describes the design and analyses of the cable stayed option for the SFOBB East Span. The following are the conclusions from that paper:

"The design combination of composite deck, shear-linked tower and splayed cable configuration represents a unique and progressive solution, which is a departure from the classical design approach of a cable-stayed bridge. The innovations in this design were developed in response to the challenges of design for the unique seismic demands and architectural requirements of this bridge site. Of particular note is the excellent performance of the shear-linked pylon design, which contrasts sharply with the conventional approach of weak-column/strong beam used in seismic design of contemporary bridges. The superior performance of the weak-beam solution allows all ductility to reside in replaceable steel links, greatly improving the reliability of the vertical load carrying tower sections. The resulting structural system improves performance over traditional solutions, and provides a new benchmark in major bridge design for cable-stayed structures in regions of extremely high seismicity."

All of the cable stay options bring potential cost savings greater than $500 million. However, they have various degrees of; environmental impacts (due to potential foundation increases in size and number and aesthetic considerations), seismic performance characteristics, adjacent contract impacts, and schedule impacts, the focus of the Phase 2 effort discussed in the remaining sections of this report was to further investigate these issues so that sound conclusions could be made to advance recommendations going forward.

Preliminary Design Development

3. PRELIMINARY DESIGN DEVELOPMENT

3.1 Objectives

The objectives of the preliminary design development effort during Phase 2 IRT work was to examine the key technical issues with respect to cable-stayed redesign alternatives to assess that no major design difficulties or EIS issues would be encountered during the final design development phase. Key issues in this stage were:

1. Determine the foundation sizes and environmental impacts
2. Confirm that seismic standards can be met with a concrete tower as proposed
3. Determine impacts to adjacent structures (Skyway and YBI)
4. Refine/confirm previous estimates of cost savings and construction schedules
5. Identify the best option(s) for further design development (to maximize cost savings and minimize schedule and project risk)

Due to the need to make a decision with respect to the redesign/re-bid options in early 2005 (expected to be January), resolving the above issues quickly became essential. To best use the limited amount of time and resources available in Phase 2 of the IRT's work, the alternatives were prioritized in the following manner.

Prioritization of Alternates

Alternate 1 was studied first, as this is the one requiring the tallest tower, largest foundations, and the highest performance demands for the towers, foundations, and interfaces. Alternate 3 was studied next, as this was initially estimated to be the one with the shortest construction schedule and the largest of potential cost savings. Also its two-tower, three-span structural configuration results in technical issues that are quite different from the single-tower, two-span Alternate 1. The foundation and seismic issues associated with Alternate 2 can be inferred from Alternate 3 due to similar tower height and foundation size. Thus, Alternate 2 was set aside initially until the design developments on Alternates 1 and 3 are sufficiently advanced. Also, Alternative 2 has an additional pier in the bay, and it is the one with the greatest potential environmental impact and thus the greatest schedule risk. Therefore, focusing first on the other two was deemed justifiable. The limitations on schedule and resources did not permit Alternate 2 to be directly developed. However, the results obtained from Alternates 1 and 3 are sufficient to draw conclusions on the key issues on Alternate 2.

3.2 Preliminary Design Development Approach

As the key objectives of this investigation were to identify the foundation impacts, interface issues, seismic performance and design demands on the tower, and to ensure sufficient flexibility during the final design development, conservative assumptions (covering a relatively wide range of possibilities) were made with respect to the following elements:

1. Roadway deck
2. Weight of the superstructure
3. Tower modeling and design checks
4. Foundation modeling, pile layouts, and design checks

5. Energy dissipation, ductility, and safety
6. Loading conditions
7. Interface issues

3.2.1 Roadway Deck

The most attractive options for the roadway deck for the cable-stayed alternates include the use of one of the following two systems

1. Precast, prestressed lightweight concrete panels with a concrete overlay:

   The concrete slab design will be based on the same stress/strain limitations used for the Skyway structure, and the overlay thickness of 40mm assumed is twice that provided on the Skyway structure. Thus, the life expectancy of the concrete deck is expected to be at least equal to that of the Skyway

2. Steel orthotropic deck with an asphaltic overlay similar to the one on SAS

Both of these deck options will provide equal performance to those elements of the SFOBB SAS design.

3.2.2 Weight of the Superstructure

The different superstructure configurations that can be considered in conjunction with the two types of roadway deck noted above consist of:

1. SS1: Steel composite superstructure using a lightweight concrete deck and Grade 50 steel for edge girders and floor beams, and 40mm overlay and concrete barriers supported by two cable planes

2. SS2: Steel composite superstructure using a lightweight concrete deck and Grade 70 steel for edge girders and floor beams, and 40mm overlay and concrete barriers supported by two cable planes

3. SS3: Steel composite superstructure using a lightweight concrete deck and Grade 70 steel for edge girders and floor beams, and 40mm overlay and concrete barriers supported by three cable planes

4. SS4: Steel superstructure using a steel orthotropic deck and Grade 70 steel for edge girders and floor beams, and 20mm overlay and steel barriers similar to SAS and supported by two cable planes

5. SS5: Steel superstructure using a steel orthotropic deck and Grade 70 steel for edge girders and floor beams, and 20mm overlay and steel barriers similar to SAS and supported by three cable planes

The weight of superstructure SS1 (heaviest option) was worked out using a combination of preliminary sizes and allowances based on past experience. The weights of the others were estimated down from SS1 based on simple proportioning. The preliminary weight estimates for the different superstructure options described above are tabulated in Table 3.1. For engineering analysis, a superstructure weight of 350 kN/m was assumed, corresponding to SS1. This (selection of the heaviest option) would produce the most aggressive seismic demands on the foundations, tower(s), cables, and the interfaces. Thus, it would also ensure the validity of conclusions from the Stage 2 design developments if any one of the cable-stayed alternatives were to be further developed.

Table 3.1: Superstructure Options and Estimated Superstructure Weights

Superstructure Item		Weights in kN/m (Per Roadway)			
		SS1	SS1	SS3	SS4 / SS5
1	Roadway Deck[9]	130	130	130	52
2	Deck Over Floorbeams	7	7	7	≈ 3
3	Steel Box Edge Girders	72	60	50	≈ 50
4	Steel Longitudinal Struts[10]	12	12	12	≈ 12
5	Steel Floorbeams[2]	38	30	20	≈ 30
6	Barriers	15	15	15	5
7	Railings	1	1	1	1
8	W/S	29	29	29	15
9	Fiberglass Panels	10	10	10	10
10	Bike Path / Ballast	36	36	36	36
Total estimated superstructure weight (kN/m)		350	330	310	215
% Weight savings based on SS1		0%	5%	10%	35% to 40%

3.2.3 Tower Modeling and Tower Design Checks

The concrete pier elements and foundation elements of the SFOBB project (including SAS and Skyway) are modeled using moment curvature relationships. This modeling provides a more flexible analytical model than the use of gross cross-sectional properties of the elements. Through the design development process, it was noted that the foundation and tower seismic demands were proportional to the tower stiffness. The tower modeling used in the preliminary analysis described in this report uses gross section properties. This selection enabled us to obtain conservative results for the tower and foundation elements in the relatively short time frame available. It must also be noted that the global bridge deflections, such as tower top and the superstructure at the deck level, are controlled more by the cable system and the end piers (such as W2). The numerous analysis iterations showed that these global deflections were not very sensitive to the tower stiffness in the normal design range.

Discussions during the design development phase with Caltrans indicated that the SAS tower design objective was to limit the tower response to the “essentially elastic” level for the SEE design seismic event. The same discussions suggested limiting concrete strains to 0.002 and rebar stains to 10% above yield. The tower design checks for the SEE seismic event were performed using these suggested strain limits of 0.002 for concrete and 110% yield strain (= 0.0023) for steel re-bars. However, it must be noted that these strain limits (performance criteria) are considerably more conservative than the SAS design criteria for its concrete piers and the steel tower (cited below):

[9] Two-way spanning variable thickness precast, prestressed deck panels with an average thickness of 10 inches. Actual thickness will depend on final weight optimized framing configuration.

[10] Includes allowances for secondary framing members

Concrete Piers: SAS Design Criteria dated 07/15/02, Section 7.11

a. Pier Concrete: 0.004 for FEE and 2/3*ultimate concrete strain for SEE
b. Pier mild steel reinforcing: 0.015 (approx. 7.3*yield strain) for FEE and 2/3*ultimate strain (taken as 2/3*0.09=0.06=29*yield strain) for SEE

Main Tower: SAS Design Criteria dated 07/15/02, Section 7.11.4

a. Max strain for the steel tower design is 4*yield in case of overload.

As the SEE event is an extreme event condition, the 110% yield strain limit on steel rebar and 0.002 strain limit on concrete used in the present tower design checks represents considerably more conservative performance criteria than used in the design of the SAS (for the steel tower and other critical concrete piers). In fact, the strain limits assumed presently for the concrete tower design checks at the SEE level are lower than those permitted in the SAS design criteria for the FEE event.

It is the IRT's opinion that the performance criteria for the concrete towers need to be refined further so the present over-conservatism can be adjusted back to a reasonable level.

3.2.4 Foundation Modeling, Pile Layouts, and Design Checks

The analytical foundation model used in the present analysis is the same as SAS. The same pile layout, pile properties, and the pile structural capacities developed for the SAS design were used for performing the design checks for the cable-stayed alternatives. These foundation design checks are based on the following:

1. Pile structural capacities used for the SAS design provided to us by TY Lin for T1 and E2 foundations
2. Pile ultimate geotechnical capacity used (or considered acceptable) for SAS design. These include:
   a. T1 Drilled Shafts: 100 MN Tension, 185 MN Compression
   b. E2 Piles: 65MN Tension and 125 MN Compression

The 185 MN geotechnical capacity used for the T1 piles is the sum of 145 MN in skin friction and 40 MN in end bearing. The 140MPa in skin friction used in SAS design is based on an assumed ultimate skin friction value of 100psi. We have also been informed that the contribution of end bearing was ignored in the original SAS design, but Caltrans is looking into shortening the SAS shaft lengths by incorporating this additional capacity.

***Considerably Higher Geotechnical Capacity Is Justifiable Based on Geotechnical Test Data*:** It must be noted that the review of the geotechnical test data for the T1 location indicates the estimated ultimate geotechnical capacities used in the SAS design are extremely conservative. This is illustrated in the following sample computation based recommended ultimate rock design strength values reported in the 30% design report (Page 12 of Section II: Geology) and the log for boring 98-2, taken within the footprint of the T1 foundation. The 30% design report recommends the following unconfined compressive strengths for the different rock types encountered:

Rock Type	Estimated Unconfined Compressive Strength q_u	Ultimate Rock Socket Side Resistance[11]
Sandstone, Low RQD	8,400 psi (57.5 MPa)	250 psi (1.52 MPa)
Sandstone, high RQD	19,500 psi (135.0 MPa)	395 psi (2.41 MPa)
Siltstone/Claystone	3,500 psi (24.0 MPa)	175 psi (1.07 MPa)
Shaft Concrete	5,750 psi (35.0 MPa)	200 psi (1.22 MPa)

Figure 3.1 is a graph extracted from AASHTO Standard Specifications that was used in obtaining the above ultimate rock socket side resistance values using the rock strength data and the strength of concrete to be used for the drilled shaft construction. The review of boring 98-2 reveals the following rock type composition along the length of the boring: Sandstone 86%, Siltstone 9%, and Claystone 5%. The RQD of Sandstone is relatively high along the shaft length. The average side resistance computed using the rock strengths far exceed the side resistance based on the shaft concrete strength. Thus, it can be concluded that the shaft side friction resistance is dictated by concrete strength and not by the rock strength. Using the 200 psi corresponding to the 5000 psi concrete strength assumed, the ultimate side resistance of the rock socket per unit length is 8.4 MN/m, and the required 140 MN can be achieved in a 16.6m shaft length rather than the 30m design length currently specified.

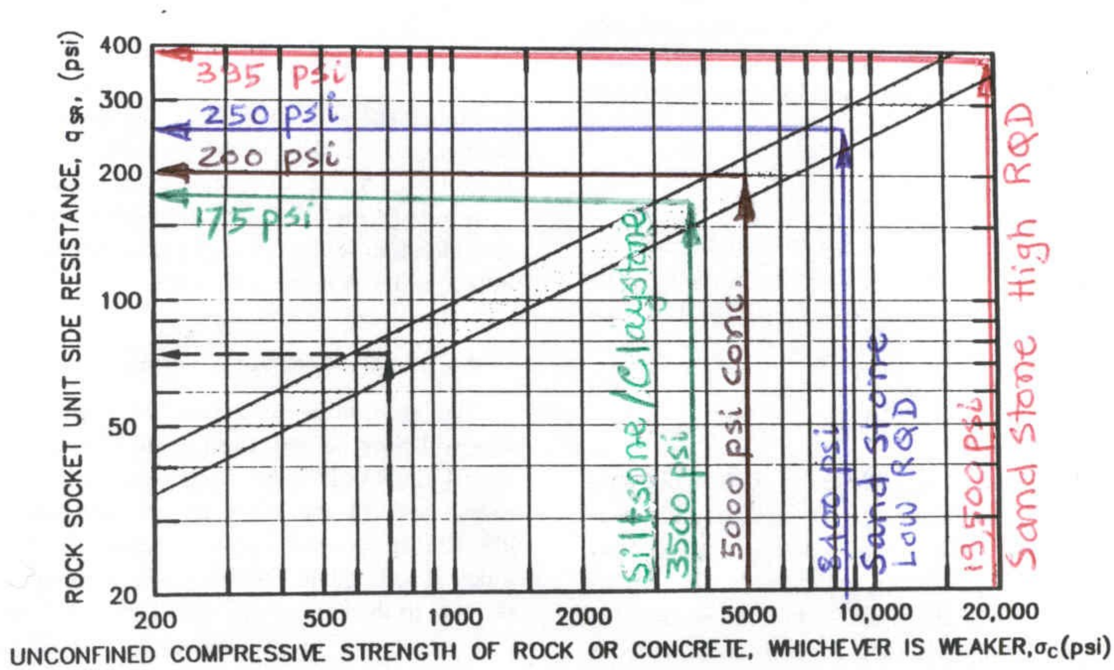


FIGURE 4.6.5.3.1A Procedure for Estimating Average Unit Shear for Smooth Wall Rock-Socketed Shafts Horvath, et al. (1983)

Figure 3.1: Ultimate side resistances (from AASHTO)

Thus it can be observed that the geotechnical shaft capacities currently used for the SEE event appear to be extremely conservative and, as illustrated later, the actual pile lengths can be shortened for the cable-stayed options.

[11] From AASHTO Standard Specifications, Figure 4.6.5.3.1A (See Figure 3.1)

3.2.5 Energy Dissipation, System Ductility and Seismic Safety

The four-legged concrete tower design is similar to the SAS tower configuration, and the preliminary analysis incorporated the same shear links as provided in the SAS design. Further, the performance criteria used in the present study for shear links are the same as the SAS. The amount of shear links could be easily adjusted as the designs are further developed.

Pier W2 is a critical pier in the SAS design and includes ductile detailing appropriate for such a critical element. For Alternate 1, we have provided additional W2 Pier columns and reduced the seismic demand per pier column. This provides a system with the same ductility level as SAS, but with much less seismic demand per column, providing an additional level of seismic safety. Alternatively, the number of W2 columns could be reduced to bring the seismic demand per column up to the same level as SAS. This is a final design issue that can be explored in the next stage of design development.

The stay cables of the cable-stayed options provide considerable stability to the tower. The tower structural behavior is considerably improved (from the essentially flagpole type behavior in the SAS system). The increased tower stability results in better tower performance under seismic loads. As discussed previously, the seismic performance criteria selected for the concrete tower design checks will enable the structure to withstand a higher magnitude seismic event with the same level of performance as the SAS design criteria or provide a better performance level than required in the current SAS design under the design SEE event.

3.2.6 Loading Conditions

Time limitations required the preliminary analysis to be based upon the one or two seismic records (form the total of six available) that would govern the design of the global elements. Based upon the experience with the SAS design, TY Lin staff picked the ground motion record 1 for the preliminary analysis. This enabled us to be able to execute a reasonable number of analysis iterations needed in the design developments within the timeframe available.

In addition to the above DL + SEE Seismic loading, the AASHTO Group I factored load combination using highway traffic loading was also used in checking the major elements of the superstructure. As described later on, seismic loading governed the superstructure design by a considerable margin, indicating that the DL and LL+I load combinations are not likely to control the final design.

3.2.7 Interface Issues

The SAS design interacts with the adjoining YBI and Skyway structures through Hinge K located west of Pier W2 and Hinge A located east of Pier E2. The interface mechanisms provided at these locations transfer the loads from one structure to the other and facilitate the necessary relative movements between the two structures. The structural systems envisioned for the cable-stayed alternatives will be designed to interact at the interfaces in the same manner. Cable-stayed alternatives were developed with options on where the transitions will be located. These transition options included:

- Keeping the transitions exactly where they are now, so no change to the Skyway design is needed
- Moving the transitions to locations that optimize the overall cost and schedule

Both of these are viable options with different advantages, and would work equally well from a technical standpoint. The key issue with respect to the interfaces is the forces and movements that the hinge devices must accommodate. This can easily be established by comparing the SAS design force and movement levels at the hinge locations to those obtained for the cable-stayed alternatives.

YBI: All of the cable-stayed design options keep the YBI interface near the existing Hinge K location. As the YBI design is still being developed, it is our opinion that any minor modifications needed could be built into the design of YBI.

Skyway: As noted previously, some of the optional layouts are developed, keeping the Skyway transition at the existing Hinge A location. For these, the only check needed is the force and movement levels in the hinge mechanisms. In this case, no Skyway design change is anticipated. For those cable-stayed options where the Skyway transition is located away from the existing Hinge A location, an evaluation of the impact to the Skyway due to the location change must be made.

Following is a summary of the possible hinge locations on the Skyway side:

- Cable-Stayed Alternates 1, Transition Option A: The cable-stayed superstructure is continued over Pier E2 up to the Hinge A location, similar to the SAS design. Thus location of the transition point is unchanged. Under this scenario, if the forces and displacements are within those for the SAS design, there is no impact to the Skyway.

- Cable-Stayed Alternate 1, Transition Option B: The Skyway structure is continued over Pier E2 to a revised hinge location west of E2. The extension length of the Skyway structure can be selected to provide the best possible scenario for the Skyway, as this is not critical to the cable-stayed design. In addition to the force levels in the hinge mechanisms, the consequences of Skyway extension by one more span must be addressed. This transition option eliminates the need for temporary piers (to support the Skyway until the main span is complete, and also offers schedule advantages. We anticipate that there are viable options for handling the impact of the revised hinge location on the Skyway design.

- Cable-Stayed Alternate 2: The Skyway transition options A and B are the same as those described for Alternate 1, with similar conclusions.

- Cable-Stayed Alternate 3: The Skyway transition location for this alternate has to be located west of Pier E3, close to the start of the steel nose section under the SAS design. In addition to the interface forces, the impact to the existing Skyway design due to the reduction in weight of the cantilever[12] must be considered. However, this can be handled relatively easily by providing a sufficient permanent ballast weight at the end of the Skyway section, or a combination of sufficient permanent ballast and a sufficient permanent reaction from the cable-stayed bridge.

Alternate 1, Transition Option B was selected for analysis, as it is the most conservative for W2 and T1 foundations[13]. It is also more conservative for E2 foundations, as the seismic shears due to a heavier extended Skyway would be considerably more that under transition Option A. This would ensure that the analysis conclusions from transition Option B with respect to the foundations, towers, global superstructure behavior, and interface forces would apply conservatively to the transition Option A.

3.3 Preliminary Design Development Process

The preliminary design development process selected was custom tailored to identify the foundation and environmental impacts, tower design potential, seismic safety issues, and interface issues in a conservative manner within the relatively short time span available. For this reason, the structural layouts, weights, and other input data used in analysis were selected to be the most conservative of the range of possibilities for each of the cable-stayed alternates. The parameters such as the superstructure weight, section properties, and the Dead Load (DL) condition used in the analysis were selected to cover the range of options discussed previously with respect to the different choices available for the next stages of design development. Final design would allow further optimization of structural elements of any of the cable-stayed alternatives.

The time span available for this investigation was not sufficient to develop computer models needed for seismic analysis from the beginning, using independently developed foundation elements and the soil-structure interaction aspects. To expedite the design development process, HNTB requested the original ADINA model files from the SAS designer, TY Lin. However, due to some logistical issues, the working arrangement for the preliminary development phase consisted of HNTB providing the bridge layout, structural information, and other parameters to TY Lin for running the analysis and providing the results to HNTB for the next iteration of preliminary design development. Following are summary descriptions of this preliminary design development approach:

1. HNTB developed models of the cable-stayed alternates where the foundations and the interface effects were represented using equivalent mass and stiffness properties. This model was used to obtain the desired DL condition, verify the different member sizes,

---

[12] Due to the elimination of the steel nose section

[13] For Option B, Pier E2 is not directly connected to the main span bridge and results in higher seismic demands on W2 and T1

and examine stability and other global issues. It was also used as a tool to examine the effects of certain design refinements on foundations, towers, and other global elements. The same basic model was also used in Live Load (LL) analysis and subsequent pushover analysis. To make the comparisons transparent, the same overall modeling arrangement, similar to that used by TY Lin's, was adopted (nodal layout, member layout, and general modeling approach) in developing HNTB's independent models.

2. HNTB then provided TY Lin the structural geometry, member sizes, and DL condition, including the cable forces, so the original SAS model could be revised to reflect the new cable-stayed layout and specific boundary conditions. TY Lin then implemented the changes in their ADINA model, ran the DL and the SEE seismic loading, and provided HNTB with analysis results.

3. HNTB also requested and obtained from TY Lin the SAS design criteria and information on the SAS design demands, as well as information on the structural capacity of the piles and E2 and W2 Pier columns.

4. HNTB refined the structural layout by evaluating the analysis results against the design criteria, seismic performance, and the capacities of the as-designed elements (piles, Piers W2 and E2, shear links etc.) to:
    a. Reduce impacts to the foundations, piles, and other as-designed elements
    b. Improve the seismic performance and safety issues
    c. Optimize the design with respect to cost and schedule

5. The above design refinement/re-analysis process was iterated about three times for each alternate to obtain the final structural layouts and the conclusions presented in this report.

Cable-Stayed Alternate 1

4. CABLE-STAYED ALTERNATE 1: 180M – 385M TWO SPAN LAYOUT

4.1 Description of Cable-Stayed Alternate 1 Structural Layout

The preliminary structure layouts shown in drawings 1 to 9 were developed following the process described previously in Section 3. The development assumptions and key features of Cable-Stayed Alternate 1 are as described in the following:

The deck weight assumed is the heaviest of the options previously listed in Table 3.1. The Cable Stay Alternate 1 was developed with two transition options on the Skyway side as the two transition options provided different advantages as noted below:

Skyway Transition Option A: This option places the cable-stayed to Skyway transition at the original Hinge A location (the same as with the existing SAS design) and has the following advantages:

1. Based on the results on the preliminary analysis[14], it has no impact on the Skyway design by inspection, and eliminates the need for re-analysis of the Skyway.

2. Avoids the sunken costs associated with the steel nose section (partly fabricated). However, this is a relatively small cost component in the overall context of the project

3. Pier and foundation E2 become a part of the cable-stayed structure and can be used to better optimize the global layout with respect to seismic performance and structural efficiency.

4. Minimal or no change to the existing hinge details

Skyway Transition Option B: This places the transition at a location west of Pier E2. The segmental concrete Skyway (typical concrete box girder superstructure) is continued a sufficient distance beyond Pier E2 on to the main span side. The advantages associated with this transition option are:

1. Faster construction schedule, as the current Skyway contractor can continue superstructure construction all the way to the new hinge location.

2. Eliminates temporary piers needed to support the steel nose section until the main span bridge is completed.

3. Eliminates the need for a third different structure type, as the main span (assumed steel composite) is transitioned to the Skyway segmental concrete.

[14] It is our understanding (per communications with T Y Lin) that the interface forces for Alternate 1 are within those the existing design can accommodate

The key technical challenges encountered in the development of Cable Stay Alternate 1 are:

- Finding an efficient structural system that can accommodate the relatively large main span to back span length ratio (this typically requires a much heavier back span than the main span).

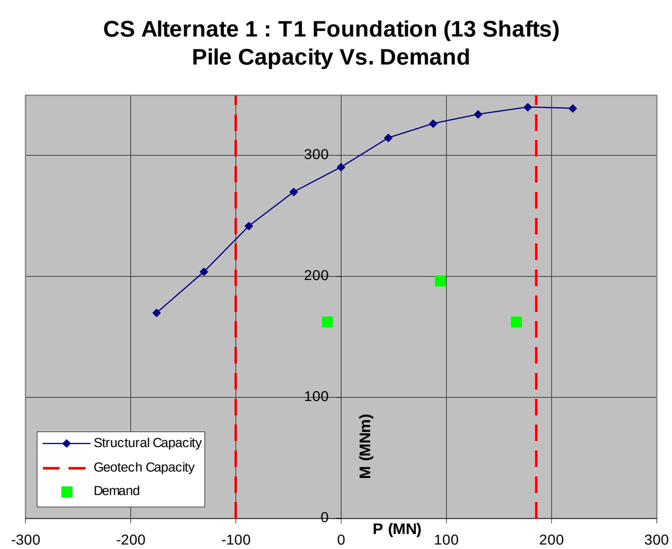
- Finding an efficient system that can resist the seismic forces due to the additional weight of the structure (when compared to SAS), without a substantial increase in tower base moments and foundation loads.

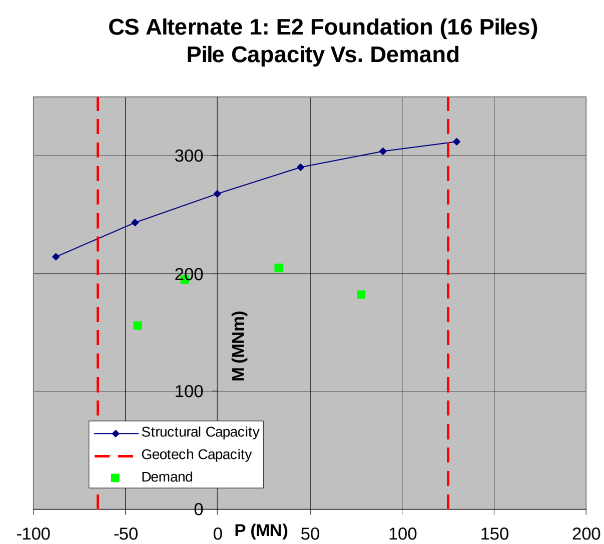
- Optimizing the structure in terms of its mass and stiffness distribution in such a way that the as-designed SAS foundations at T1 and E2 are sufficient. (The ability to use the as-designed SAS foundations[15] provides substantial cost and schedule advantages – discussed later).

An optimal solution to these three challenges was found by concentrating the additional weight of the heavier back span within a limited region at Pier W2, and then providing additional seismic capacity at W2 by adding extra pier columns. These additional pier columns not only carry the additional locally concentrated weight, but also provide a direct load path for transferring the seismic forces to the bedrock in a highly cost effective manner. This in turn reduces the seismic demands on the foundations T1 and E2 in the bay, where the costs of the foundations are very high relative to the cost of additional columns at Pier W2. The overall structural system also provides a high level of seismic safety by reducing tower demands. Additional tower stability is provided by the back stay cables anchored to the deck at Pier W2.

4.2 Results of Analysis and Design Checks (for Layout Shown In Drawings)

1. **Foundations:** The SAS foundations can be used as-is for Cable-Stayed Alternate 1. The following graphs show the demand plotted against the capacity for the drilled shafts at T1 and driven piles at E2. The pile capacities have been computed based on the same design criteria and design data as used in the SAS design.





15 See the section 4.2(1)

From the graphs, it is shown that the drilled shafts at T1 and piles at E2 have the following additional capacities:

T1 Drilled Shafts: Approximately 48% additional capacity available

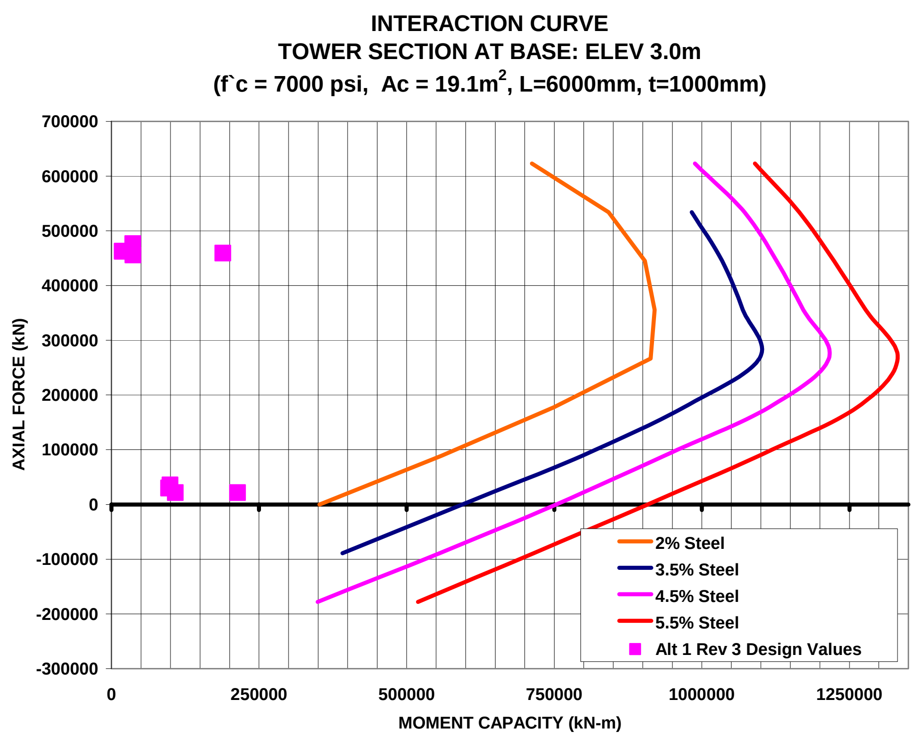
- Structural = 48%
- Geotechnical = 12% (The IRT believes the geotechnical capacity can be equal to or exceed the structural capacity based on rock strength data, see section 3.2.4

E2 Piles: Approximately 25% additional capacity available

- Structural = 25%
- Geotechnical = 40%

These additional pile capacities provide a considerable margin of design contingency. The existing pile caps can also be reused as-is (or with minor modifications) by providing a tower base plinth for load distribution. This will be done as a part of the final tower design development. Further, we have also verified through a preliminary pushover analysis (performed in the transverse direction) that the tower legs yield prior to the drilled shafts by a wide margin, and that the typical 1.5 capacity ratio can be met.

2. **Concrete Tower:** The following graph shows the tower base demand for the controlling seismic loads plotted against the capacity of the tower legs based on 0.002 strain level in concrete and first yield of rebar obtained from Caltrans' X-Section Program.



The above graph indicates that the tower legs can meet the seismic demand under the very stringent criteria adopted for the check, and have excess capacity allowing for further design optimization (and reduced seismic demands). This verifies that the concrete towers can be designed to meet or exceed the SAS/SFOBB seismic design and performance criteria.

3. **Tower Shear Links:** The same shear link properties and shear link placement as the SAS was assumed for Cable-Stayed Alternate 1. The analysis results show that the performance of the shear links is within the SAS seismic design criteria.

Shear Link Orientation	Shear Link Plastic Rotations (Radians)	
	Cable-Stayed Alternate 1	Limiting Rotation per SAS Design Criteria
Longitudinal	0.065	0.08
Transverse	0.030	

4. **W2 and E2 Pier Columns:** The seismic performance of the W2 and E2 Pier columns for the cable-stay alternatives can be verified relatively quickly by comparing the moment demand for the cable-stayed with those for the SAS design. This provides a firm verification that the pier columns can provide the same level of seismic performance as incorporated into the SAS design. The following tables compare the maximum demand per pier column at Pier W2 and per pier column at Pier E2, relative to the corresponding SAS design demands.

   Pier W2 - Maximum design demand per pier column:

	Axial MN	L-Mom MNm	T-Mom MNm
SAS	170 / -100	300	230
Cable-Stayed Alternate 1	73 / -62	237	117

   Pier E2 - Maximum design demand per pier column:

	Axial MN	Mom MNm
SAS	120 / -30	800
Cable-Stayed Alternate 1	170 / 30	880

The demands on pier columns at W2 for Cable-Stayed Alternate 1 are well below the SAS levels. The 10% higher moment demand on E2 is compensated by the beneficial effects of the increase in axial loads (especially the elimination of tension). Furthermore, these moments for the cable-stayed alternate were obtained for the worst-case scenario for this option. It is expected that the E2 moments can be reduced to the SAS levels through further design refinements.

5. **Interface Forces:** The governing forces at the Skyway and YBI interfaces are listed below:

Interface Forces and Movements - Cable-Stayed Alternate 1				
		Forces		Movement
		Transverse Shear (MN)	Vertical Shear (MN)	Longitudinal Displacement (mm)
YBI (Hinge K)	Cable-Stayed Alternate 1	18	41	1102
	SAS	16	74	1285
Skyway (Hinge A)	Cable-Stayed Alternate 1	22	23	1370
	SAS	16	32	1170

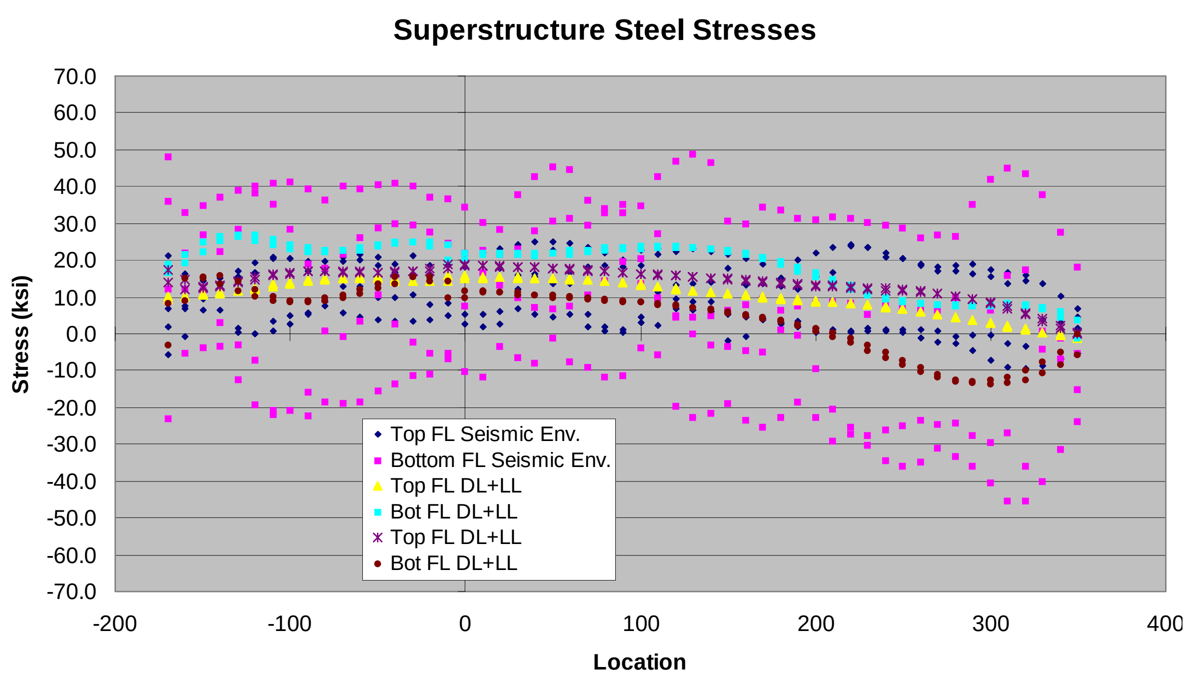
The table shows that interface forces and movements for the Cable-Stayed Alternate 1 are roughly the same as for the SAS. It is reasonable to expect that the existing hinge mechanism designs could be used for the Cable-Stayed Alternate 1 with little or no change.

6. **Global Superstructure:** The following stress plot illustrates the longitudinal girder stresses for:

- DL+SEE Seismic
- Factored AASHTO Group I loading (DL + LL+I)

From this plot the following can be concluded:

1. The stresses are within allowable range
2. Seismic load case governs the design
3. The girder section can be further reduced if Grade 70 steel is used



4.3 Conclusions of the Technical Analysis of Cable-Stayed Alternative 1

1. **General:** The analysis is based on conservative assumptions with respect to key elements such as the superstructure weight, tower weight, and tower stiffness. The demands for the foundations and towers during the next stage of design development are expected to be lower than those predicted at this stage.

2. **Foundations:** The analysis shows that the existing T1 and E2 foundations can be used as-is for Alternate 1. Furthermore, there is additional reserve pile capacity of nearly 48% at T1 and 25% at E2. It is hard to anticipate a reason for needing more piles based on the analysis data. However, should additional capacity be needed, additional piles can be added without increasing the existing foundation footprints.

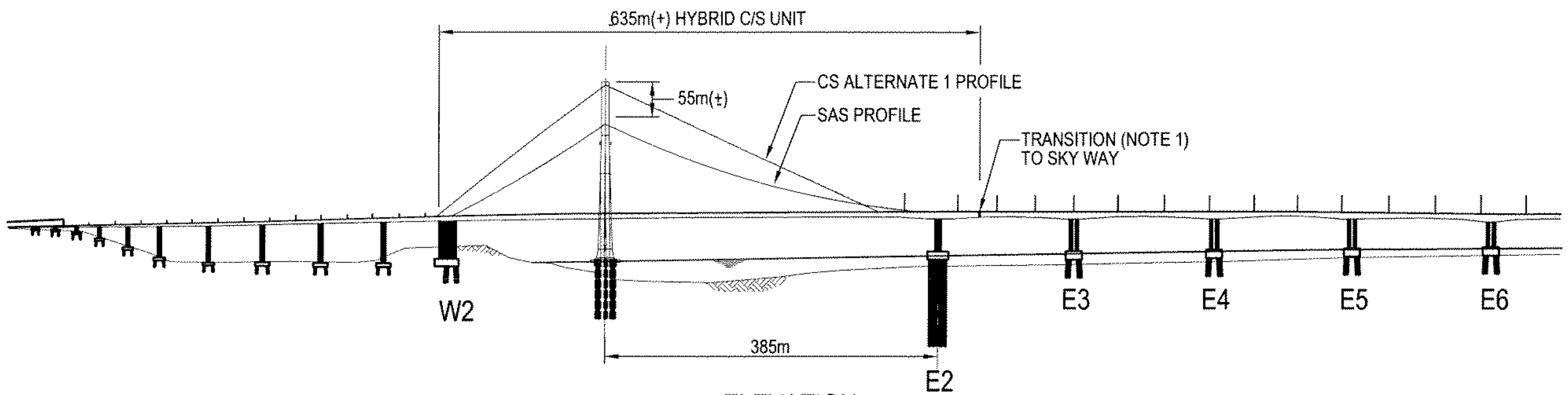
3. **Seismic Performance:** Seismic performance levels specified in the SAS design criteria can be met or exceeded for all of the elements examined. This includes meeting the strain levels with foundation elements, towers, piers, superstructure, shear links, and all other global elements that were the focus of this preliminary design development.

4. **Tower Design:** The concrete towers can be designed to meet the seismic performance requirements of the project using less than 4% rebar steel as required by ATC-32. Also, the limits on tower concrete and steel strains assumed for the present study show that the tower can be designed to a seismic performance standard equal to or exceeding those adopted for the SAS tower design.

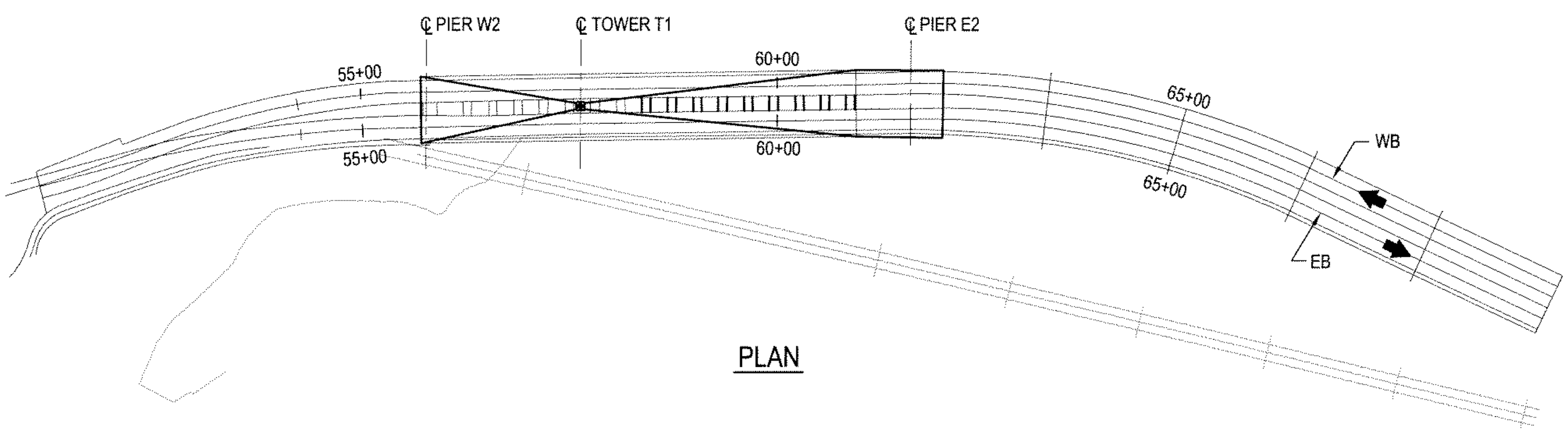
5. **Impacts to YBI and Skyway Designs:** The Transition Option A allows elimination of the design impacts to Skyway. The design impacts to YBI are minimal and can be readily incorporated in to the design. For Transition Option B, we believe that feasible solutions exist.

6. **Interface Forces and Movements:** The analysis results show that the existing hinge design and details can be used with little or no change.

M:\39442-SFOBB\DWG\ALTERNATE 1\2 SPAN GPE.DWG 11/17/04 16:08



ELEVATION



PLAN

NOTE:

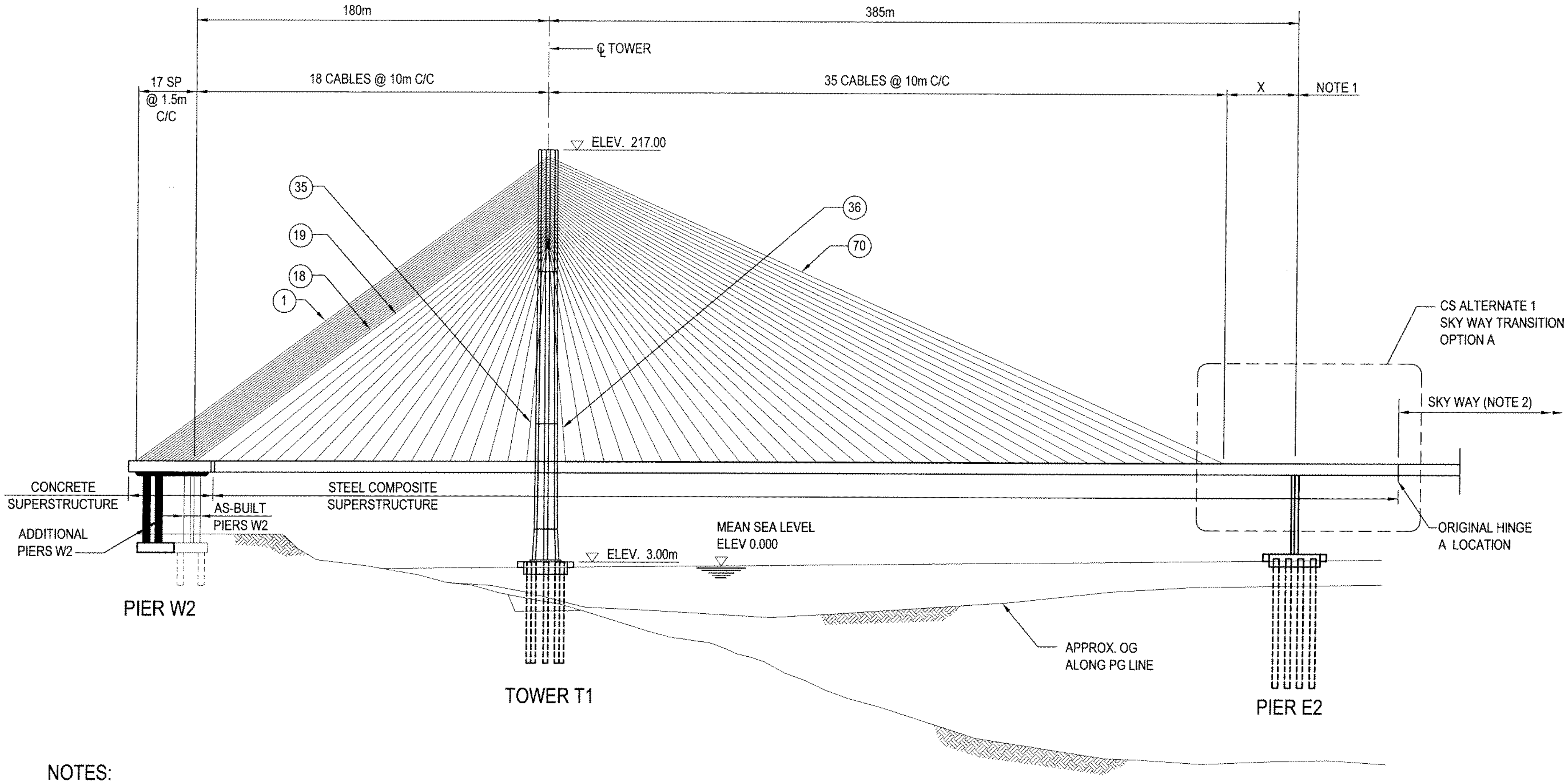
1. SEE BRIDGE ELEVATION SHEETS 2 AND 3 FOR SKY WAY TRANSITION OPTIONS A AND B.

Sheet 1 of 9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 1: 180m-385m TWO SPAN LAYOUT
GENERAL PLAN & ELEVATION



NOTES:

1. THE NUMBER OF CABLES AND TOWER HEIGHT TO BE REFINED BASED ON THE OPTIMAL DISTANCE X (TO BE DETERMINED IN NEXT STAGE OF DEVELOPMENT)
2. HINGE LOCATION: PLACED TO MATCH EXISTING HINGE A LOCATION. CS SUPER STRUCTURE BEYOND PIER E2 TRANSIT TO MATCH EXISTING INTERFACE DETAILS.
3. FOR SITE PLAN AND DETAILS OF PIER W2, SEE SHEET 6 OF 7.

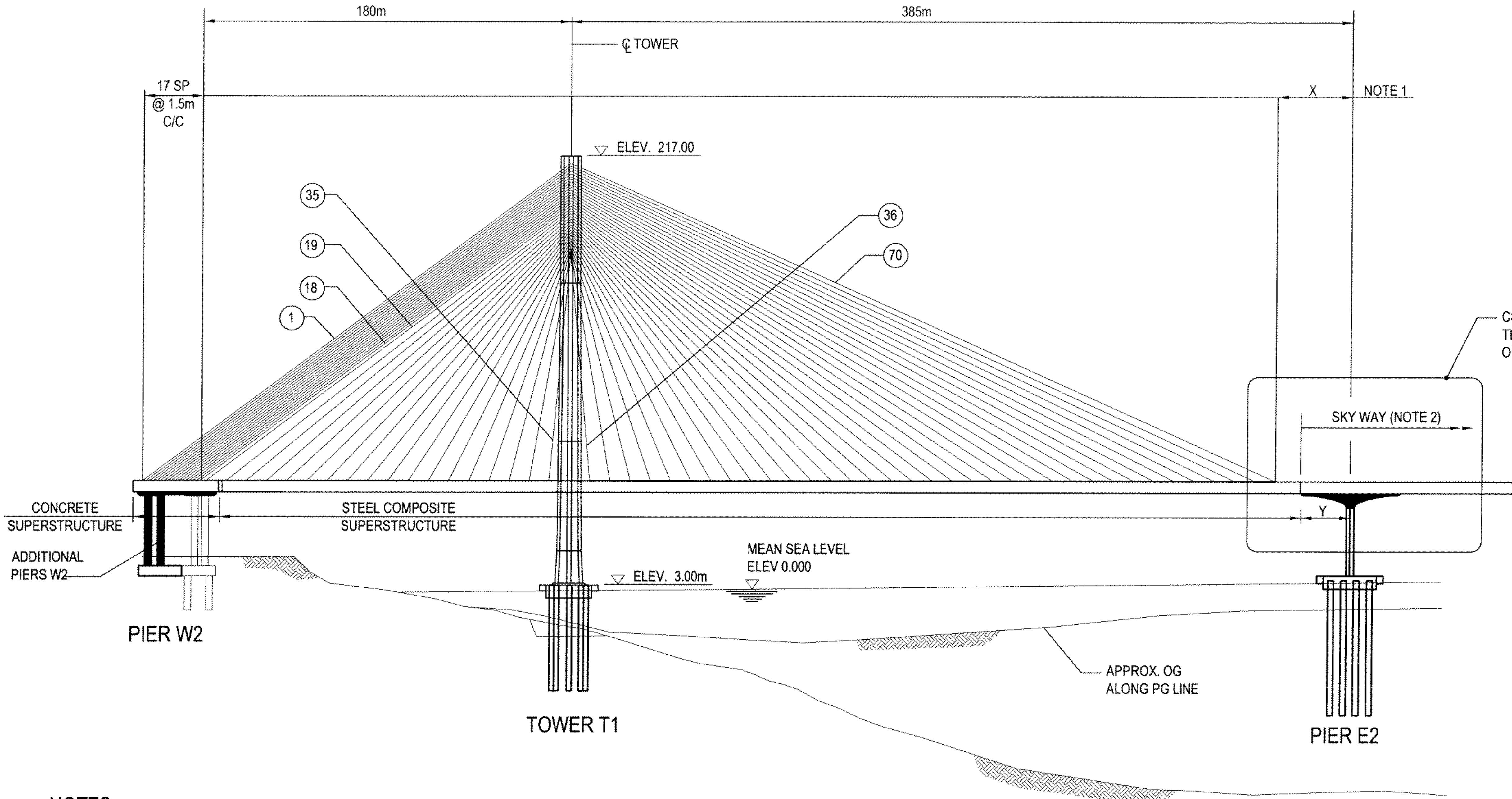
M:\39442-SFOBB\DWG\ALTERNATE 1\2 SPAN ELEVATIONA.DWG 11/17/04 16:06

Sheet 2 of 9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 1: 180m - 385m TWO SPAN LAYOUT
BRIDGE ELEVATION



NOTES:

1. THE NUMBER OF CABLES AND TOWER HEIGHT TO BE REFINED BASED ON THE OPTIMAL DISTANCE X (TO BE DETERMINED IN NEXT STAGE OF DEVELOPMENT)

2. TRANSITION HINGE LOCATED ON WEST SIDE OF PIER E2. DISTANCE Y TO BE REFINED TO SUIT DIST X (NOTE 1) AND SKYWAY DESIGN OPTIMIZATION.

3. FOR SITE PLAN AND DETAILS OF PIER W2, SEE SHEET 6 OF 7.

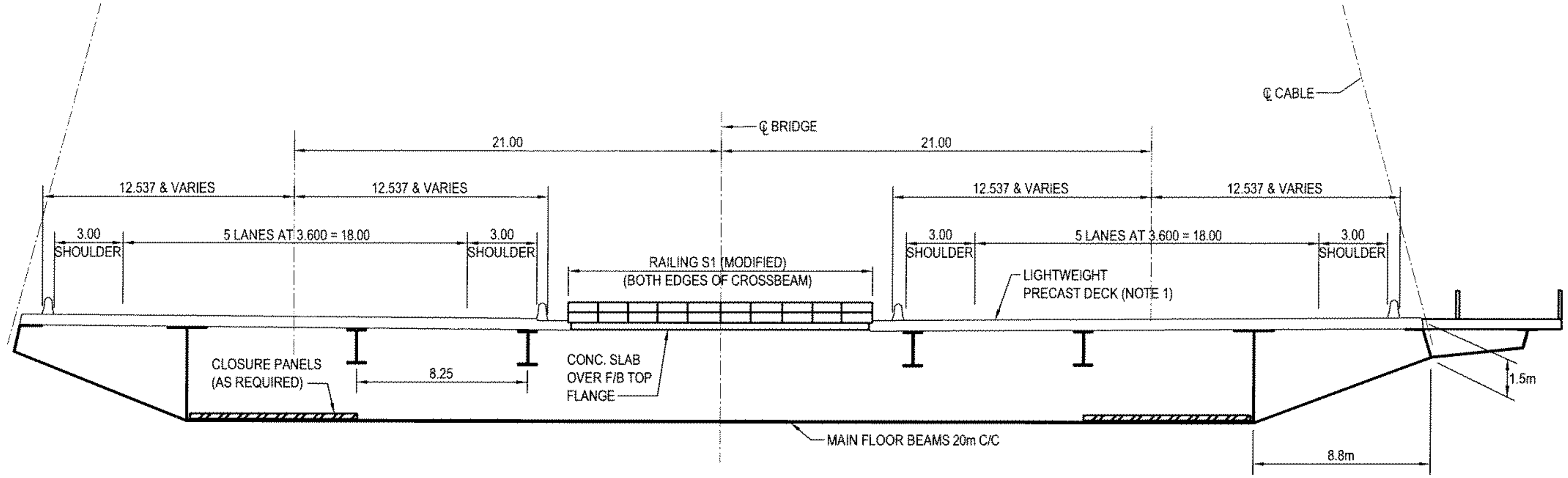
Sheet 3 of 9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

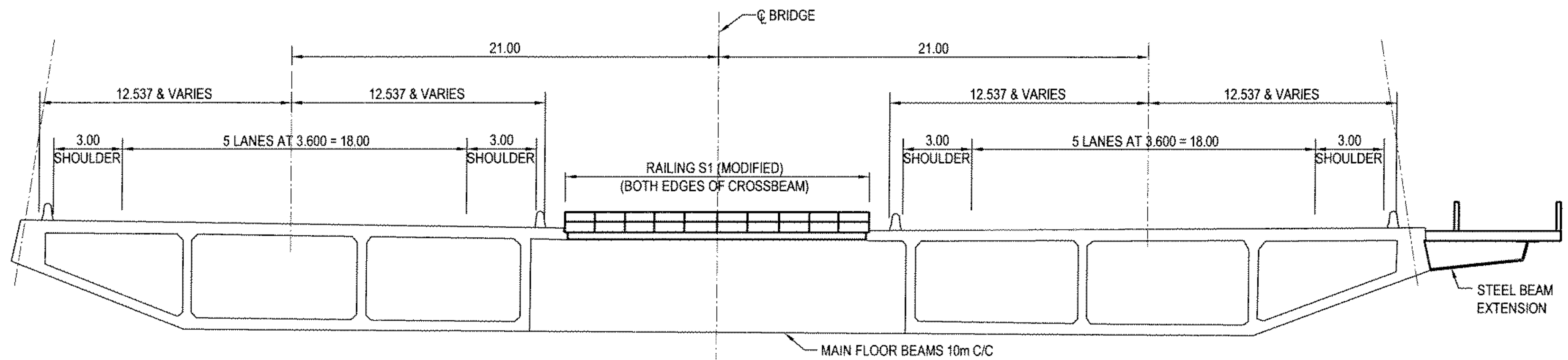
HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 1: 180m - 385m TWO SPAN LAYOUT
BRIDGE ELEVATION

M:\39442--SFOBB\DWG\ALTERNATE 1\2 SPAN ELEVATIONB.DWG 11/17/04 16:07



STEEL COMPOSITE SUPERSTRUCTURE SECTION



CONCRETE SUPERSTRUCTURE SECTION
(NOTE 2)

NOTES:

1. CONCRETE DECK ASSUMED FOR PRELIMINARY DESIGN (HEAVIEST OPTION). STEEL ORTHOTROPIC DECK CAN ALSO BE USED IN PLACE OF CONCRETE DECK. FOR THE CONCRETE DECK OPTION SHOWN, DECK SLAB DETAILS AND SECONDARY FLOOR FRAMING ARE NOT SHOWN.
2. BALLAST CONCRETE NOT SHOWN.
3. SUPERSTRUCTURE CABLE ANCHORAGE LOCATIONS SHOWN ARE SCHEMATIC.

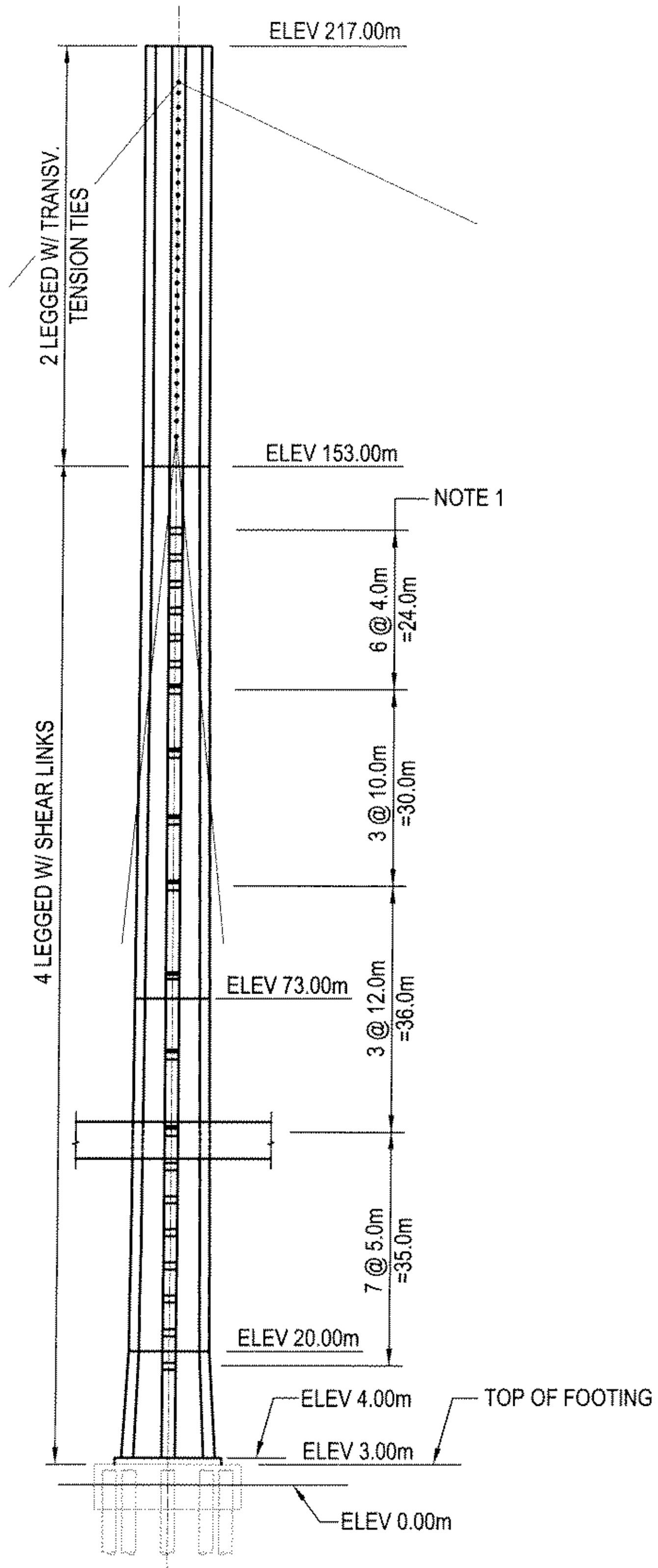
Sheet 4 of 9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

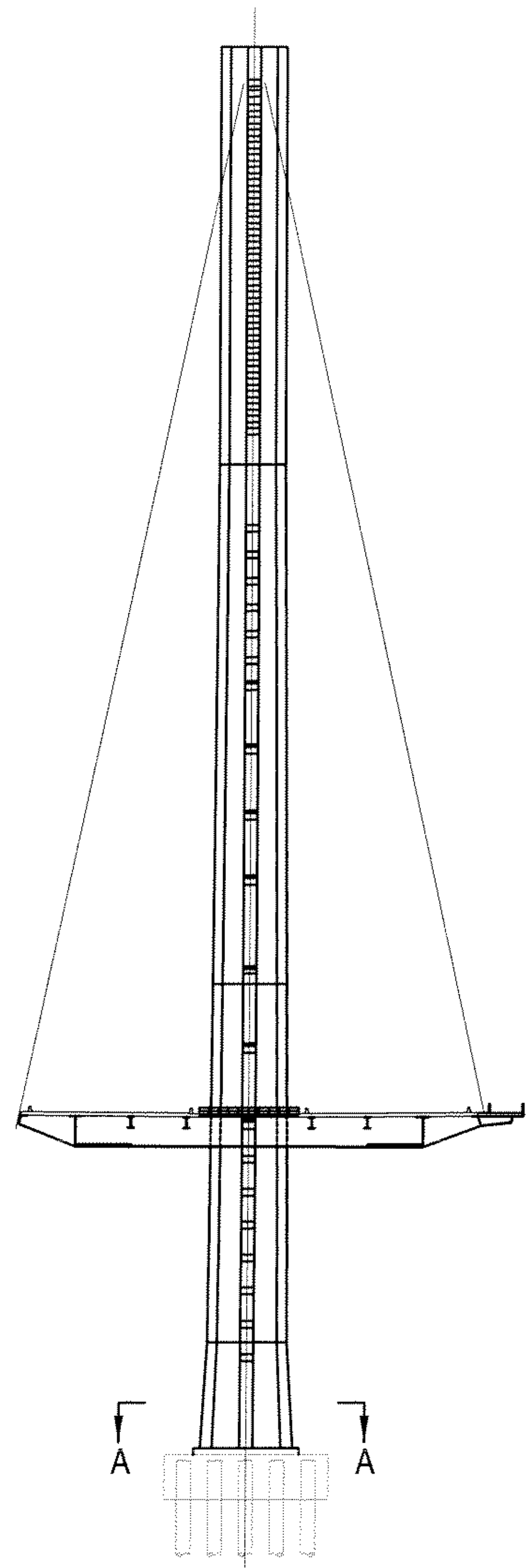
HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 1: 180m-385 TWO SPAN LAYOUT
SUPERSTRUCTURE CROSS SECTIONS

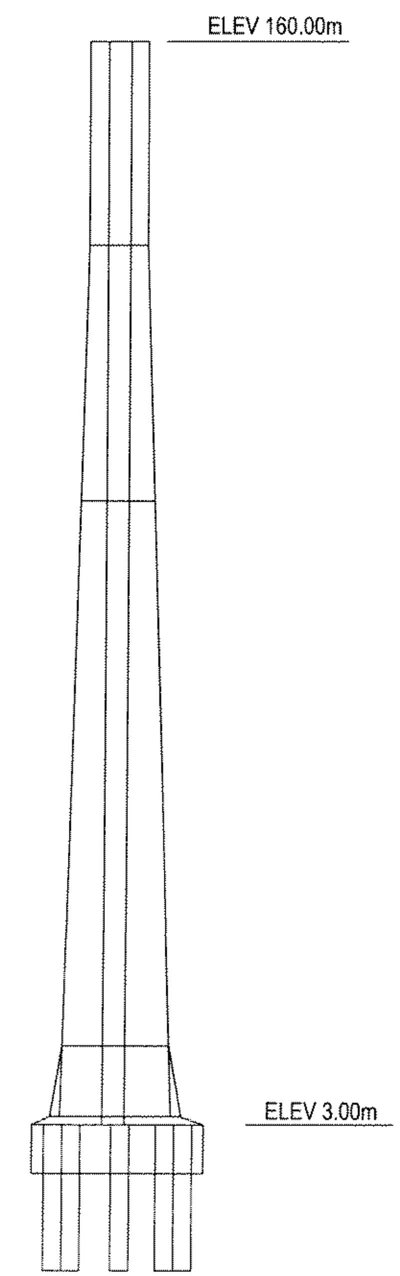
M:\39442-SFOBB\DWG\ALTERNATE 1\2 SPAN SUPER SECTION.DWG 11/17/04 16:09



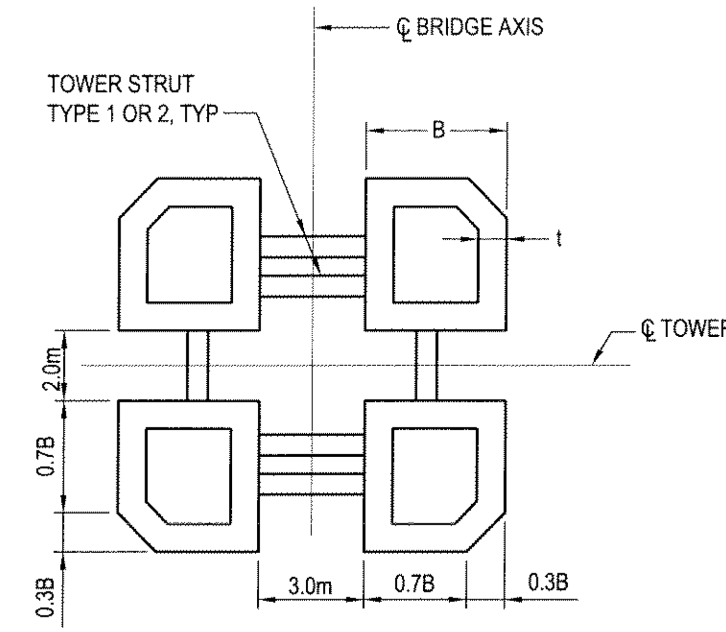
LONGITUDINAL ELEVATION
SCALE: 1:500



TRANSVERSE ELEVATION
SCALE: 1:500



TRANSVERSE ELEVATION - SAS
(FOR COMPARISON)
1:500



TOWER SECTION A-A

TOWER SECTION TABLE		
Elev.	t	B
3.0m TO 20.0m	1.0m	Varies 6.0m TO 5.0m
20.0m TO 73.0m	1.0m	Varies 5.0m TO 4.4m
73.0m TO 153.0m	0.8m	Varies 4.4m TO 4.0m
153.0m TO 217.0m	0.6m	4.0m

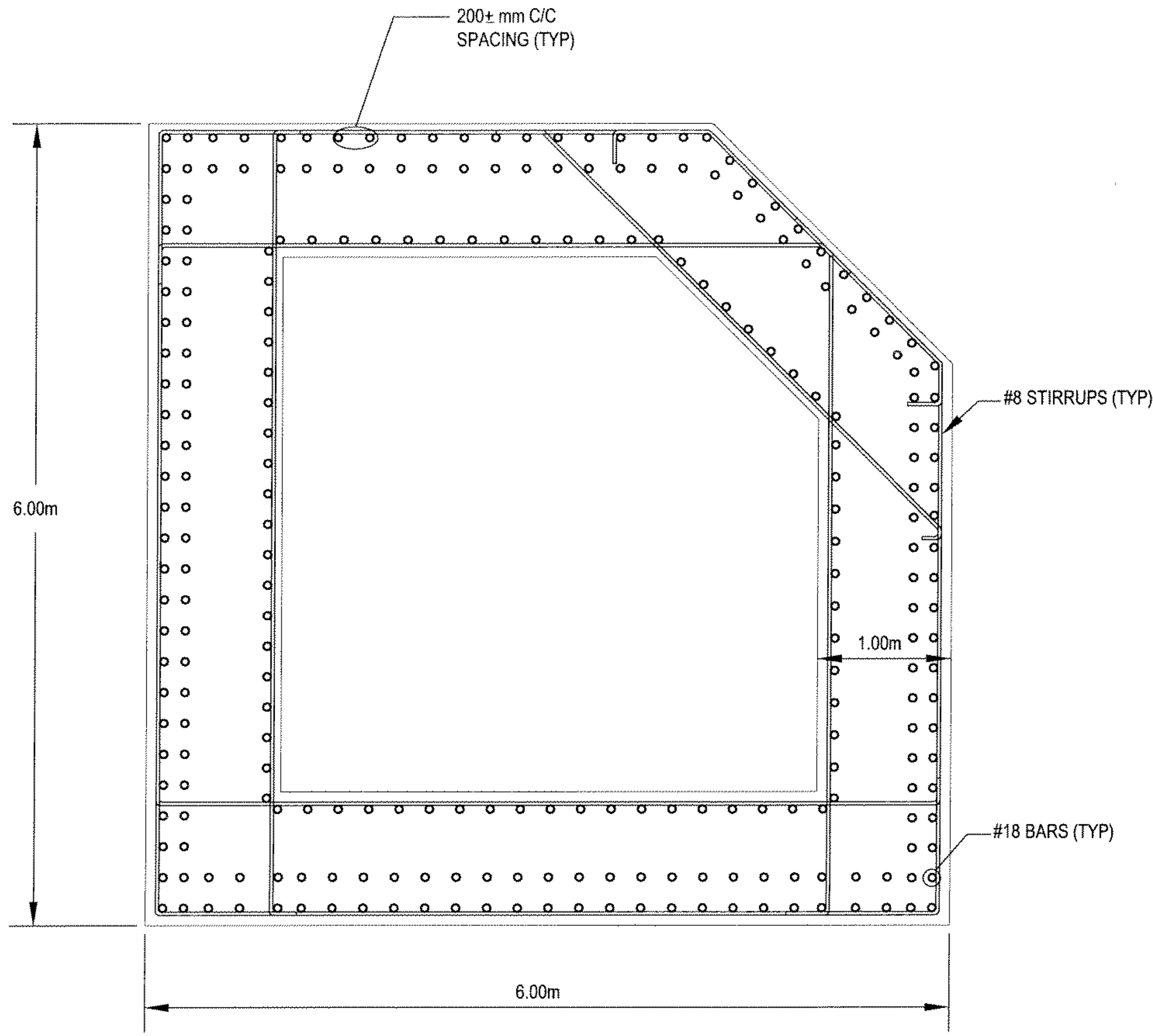
NOTE:
1. SHEAR LINKS SAME AS SAS.

Sheet 5 of 9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 1: 180m-385m TWO SPAN LAYOUT
TOWER ELEVATIONS & SECTION



TOWER LEG SECTION AT BASE

M:\39442-SFOBB\DWG\ALTERNATE 1\2 SPAN TOWERSECTBASE.DWG 11/17/04 16:09

NOTES:

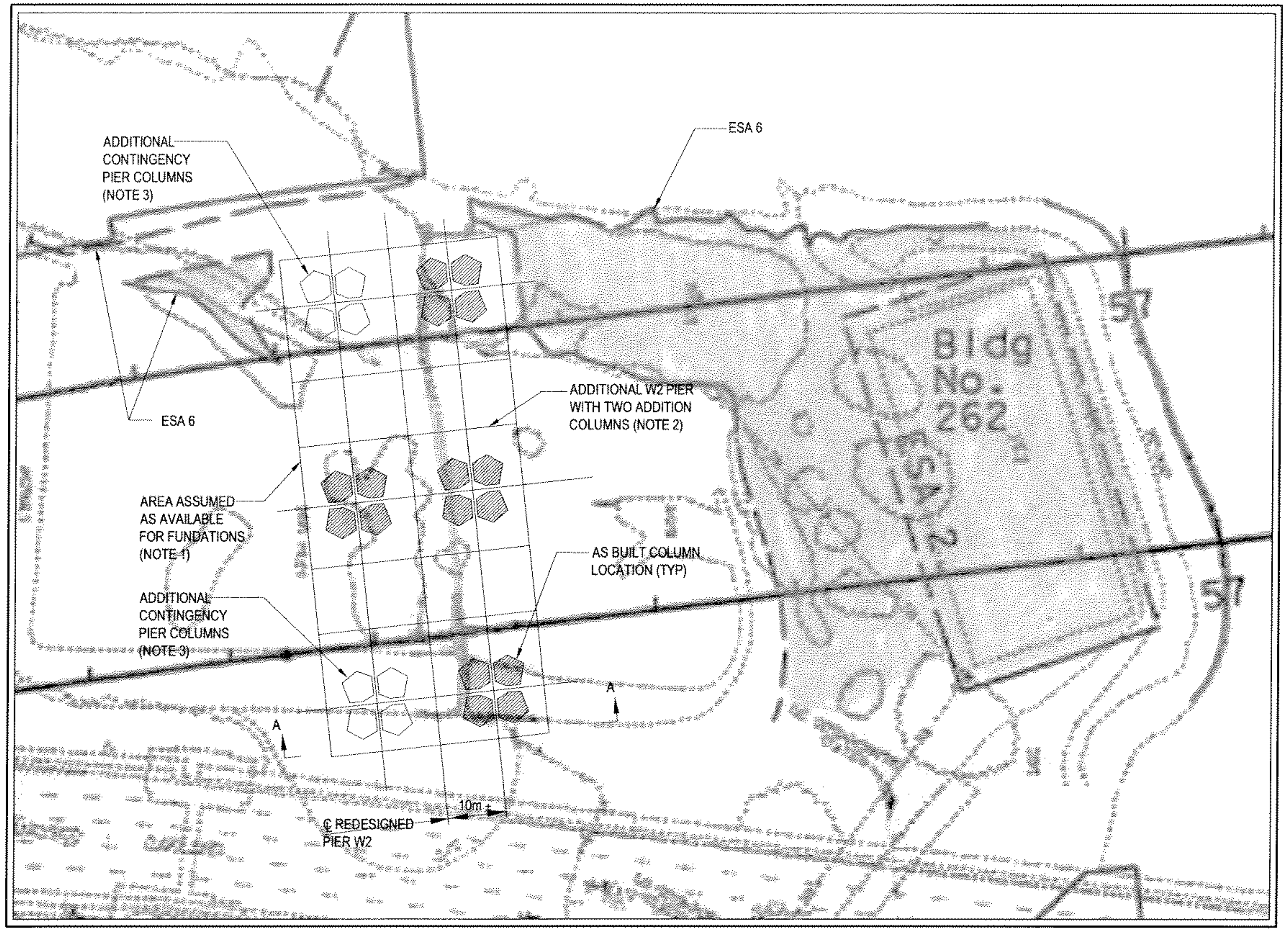
1. CROSS SECTION AND REBAR LAYOUT SHOWN ARE SCHEMATIC AND FOR PRELIMINARY DESIGN PURPOSES ONLY.
2. SECTION CONTAINS AN ESTIMATED MAIN VERTICAL 3.5% STEEL OVER THE SECTION.
3. SHEAR STIRRUPS SHOWN ARE SCHEMATIC (SEISMIC TIES NOT SHOWN).

Sheet 6 of 9

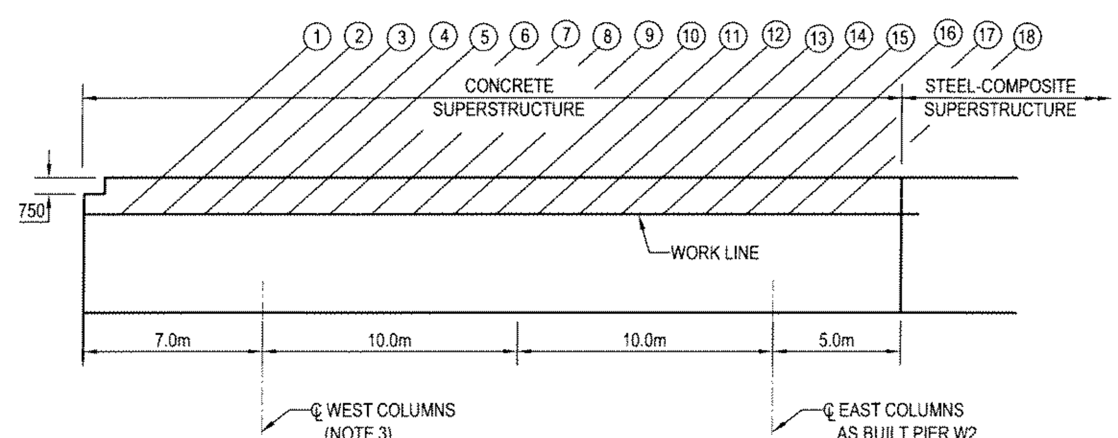
SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 1: 180m-385m TWO SPAN LAYOUT
SCHEMATIC TOWER SECTION AT BASE



PIER W2 - SITE MAP



SCHEMATIC SUPERSTRUCTURE LAYOUT AT W2

NOTES:

1. BASED ON AVAILABLE DATA.
2. LOCATION, SHAPE AND SIZE IS FLEXIBLE.
3. NOT EXPECTED TO BE NEEDED BASED ON PRESENT EVALUATION. KEEPING THESE PROVIDES ADDITIONAL CONSERVATISM.

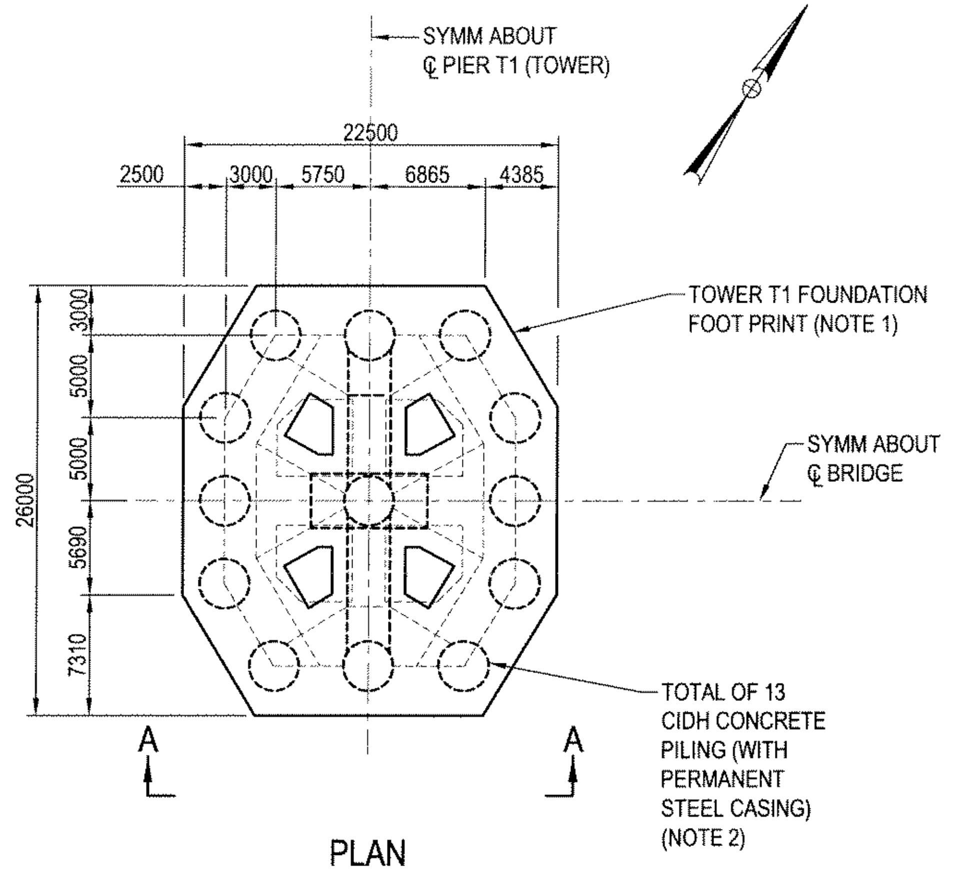
Sheet 7 of 9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

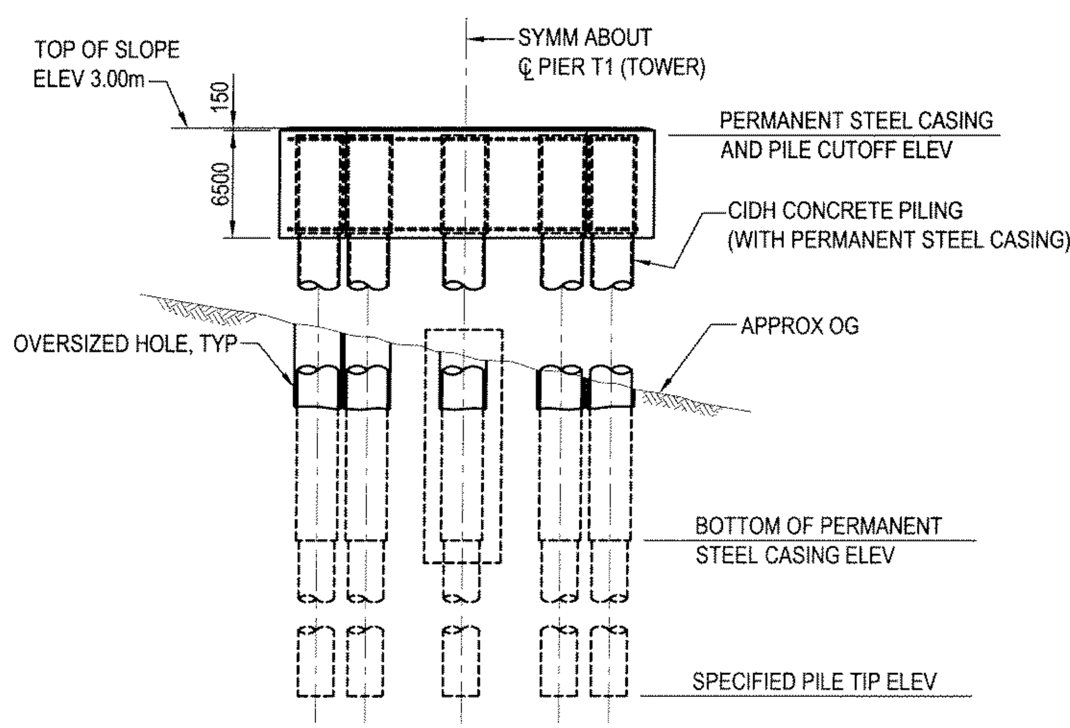
HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 1: 180m-385m TWO SPAN LAYOUT
REDESIGNED PIER W2

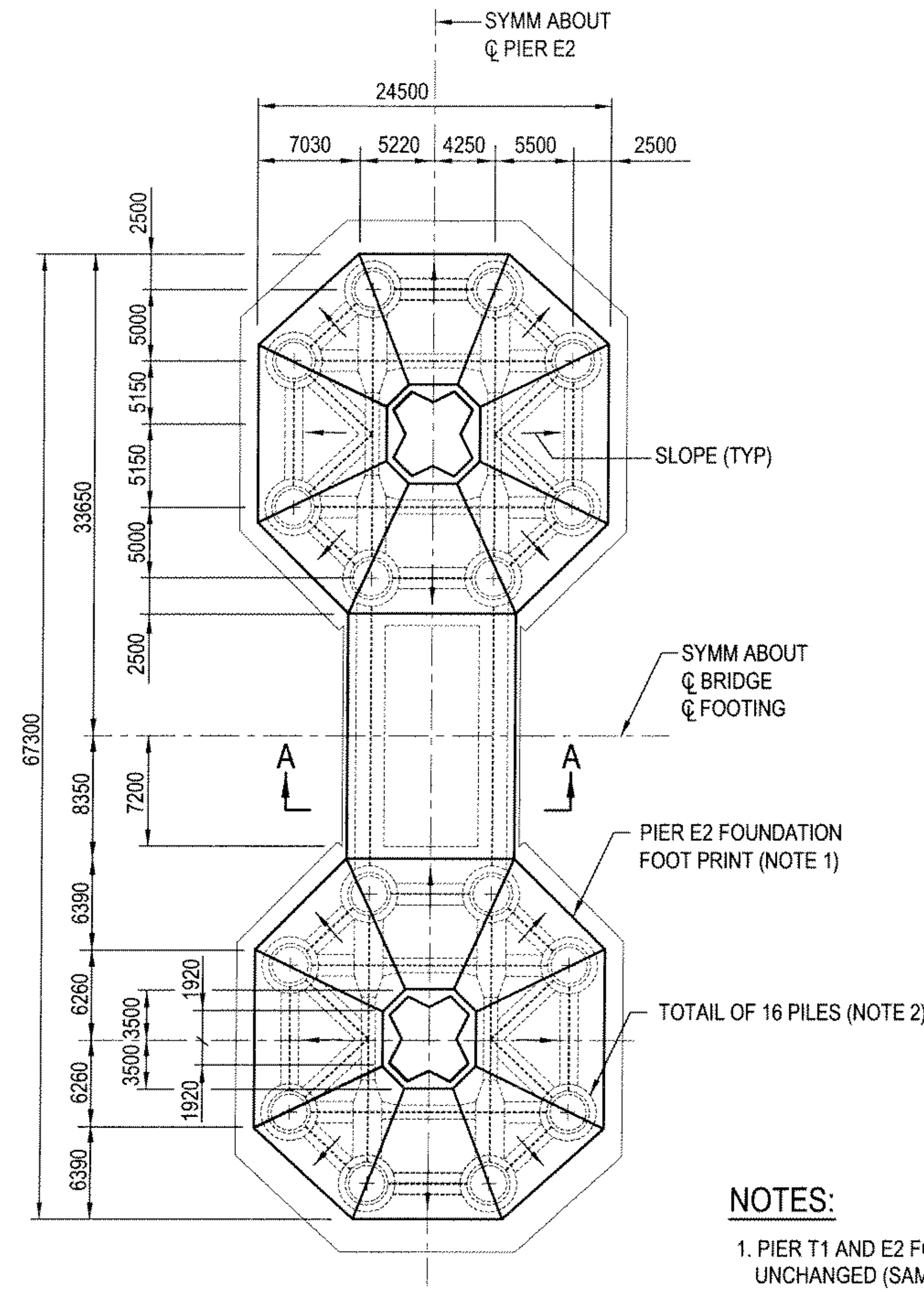
M:\39442-SFOBB\DWG\ALTERNATE 1\2 SPAN W2 DETS.DWG 11/17/04 16:11



PLAN



ELEVATION A-A



PLAN

NOTES:

1. PIER T1 AND E2 FOUNDATION FOOT PRINTS TO REMAIN UNCHANGED. UNCHANGED (SAME AS SAS)
2. PIER T1 AND E2 EXISTING PILE LAYOUTS AND DESIGN DETAILS TO REMAIN UNCHANGED (PER PRELIMINARY ANALYSIS)

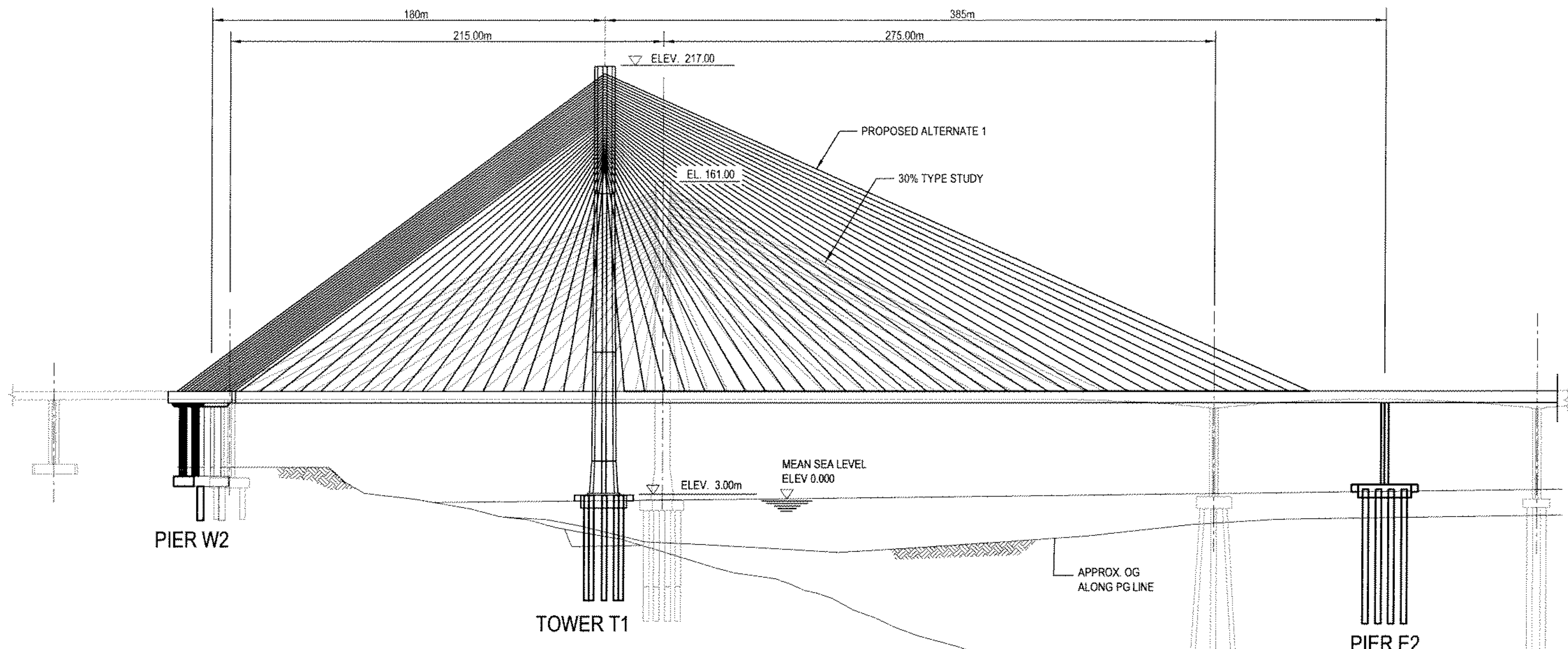
Sheet 8 of 9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

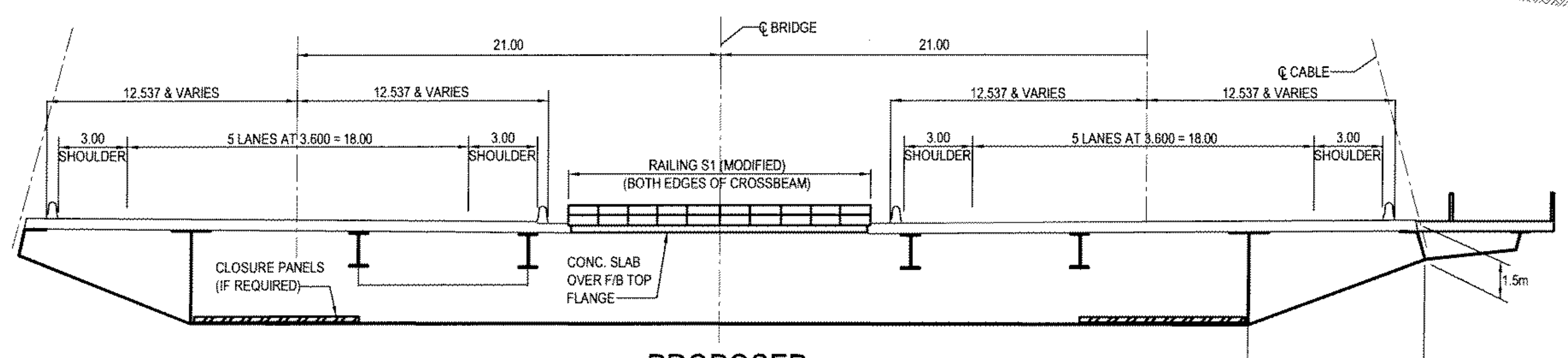
HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 1: 180m-385m TWO SPAN LAYOUT
T1 & E2 FOUNDATION LAYOUTS

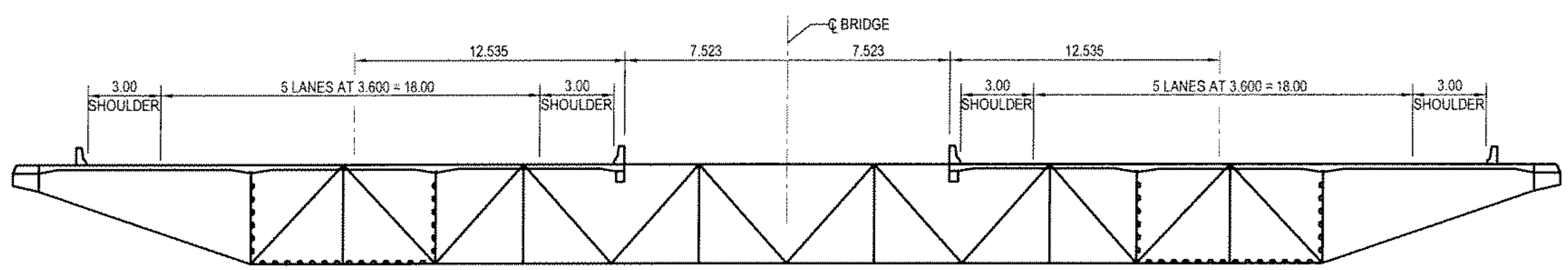
M:\39442-SFOBB\DWG\ALTERNATE 1\2 SPAN FOUNDATION.DWG 11/17/04 16:08



ELEVATION



PROPOSED



30% TYPE STUDY CROSS SECTIONS

M:\39442-SFOBB\DWG\ALTERNATE 1\EXISTELEV1.DWG 11/17/04 16:12

Sheet 9 of 9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 1: 180m-385m 2 SPAN LAYOUT
COMPARISON WITH 30% TYPE STUDY

Cable-Stayed Alternate 2

5. CABLE-STAYED ALTERNATE 2 180M – 225M TWO SPAN LAYOUT

5.1 Description of Cable-Stayed Alternate 2 Structural Layout

Cable-Stayed Alternate 2 is similar in the span arrangement to that of Alternate 1, and its tower height and foundation loads are similar to those of Alternate 3. The preliminary structure layout for Alternate 2 shown in Drawings 1 to 6 was developed based on judgment and experience gained from design development results for Alternates 1 and 3. This alternate requires an additional Pier E2A in the bay. The assumptions and key features of the CS Alternate 2 are as described in the following:

The deck weight assumed is the heaviest of the options previously listed in Table 2.1. The CS Alternate 2 was developed with two transition options on the Skyway side, as the two transition options provided different advantages as noted below:

Skyway Transition Option A: This places the cable-stayed unit to the Skyway transition at the original Hinge A location (same as with the existing SAS design), and has the following advantages:

1. Based on the results on the preliminary analysis[16], it should have no impact on the Skyway design by inspection, and eliminates the re-analysis of the Skyway.

2. Avoids the sunken costs associated with the steel nose section (partly fabricated). However, this is a relatively small cost component in the overall scheme.

3. The foundations E2A and E2 become parts of the cable-stayed unit and can be used to better optimize the global layout with respect to seismic performance and structural efficiency.

4. Minimal or no change to the existing hinge details are expected.

Skyway Transition Option B: This places the transition at a location west of pier E2A. The segmental concrete Skyway (typical concrete box girder superstructure) is continued a sufficient distance beyond Pier E2A onto the main span side. The advantages associated with this transition option are:

1. Faster construction schedule, as the current Skyway contractor can continue superstructure construction all the way to the new hinge location.

2. Eliminates temporary piers needed to support the steel nose section until main span bridge is completed.

3. Eliminates the need for a third different structure type in the middle, as the main span (assumed steel composite) is transitioned to Skyway segmental concrete.

---

[16] It is our understanding (per communications with TY Lin) that the interface forces for Alternate 1 are within those the existing design can accommodate and we anticipate the forces for Alternative 2 to be within the same range.

The back span and the main span for this option are balanced about the tower, and this alternate is a simpler design for that reason. It is expected that for Option A, foundations at T1 and E2 (E2A would be similar to E2) could be reduced approximately by 40 to 50% of their size, compared to the SAS. For Option B, foundation T1 could be reduced by 40 to 50% compared to SAS and piers E2 and E2A would be similar to that at E3.

5.2 Results of Analysis & Design Checks (for Layout Shown In Drawings)

No direct design checks were performed for this alternate: Alternate 1 was studied first, as this required the tallest tower, largest foundations, and the highest performance demands for the towers, foundations, and interfaces. Alternate 3 was studied next, as this was initially estimated to be the one with the shortest construction schedule and the largest potential cost savings. Also, its two-tower, three-span structural configuration results in technical issues that are quite different from the single-tower, two-span Alternate 1. The foundation and seismic issues associated with Alternate 2 can be inferred from Alternate 3, due to similar tower height and foundation size. Thus, Alternate 2 was set aside initially until the design developments on Alternates 1 and 3 are sufficiently advanced. Also, Alternative 2 requires an additional pier in the bay, and thus is the one with the greatest potential environmental impact and therefore the greatest schedule risk. Thus, focusing first on the other two was deemed justifiable. The limitations on schedule and resources did not permit Alternate 2 to be directly developed. However, results obtained from Alternates 1 and 3 are sufficient to draw conclusions on key issues of Alternate 2.

5.3 Conclusions for the Cable-Stayed Alternative 2

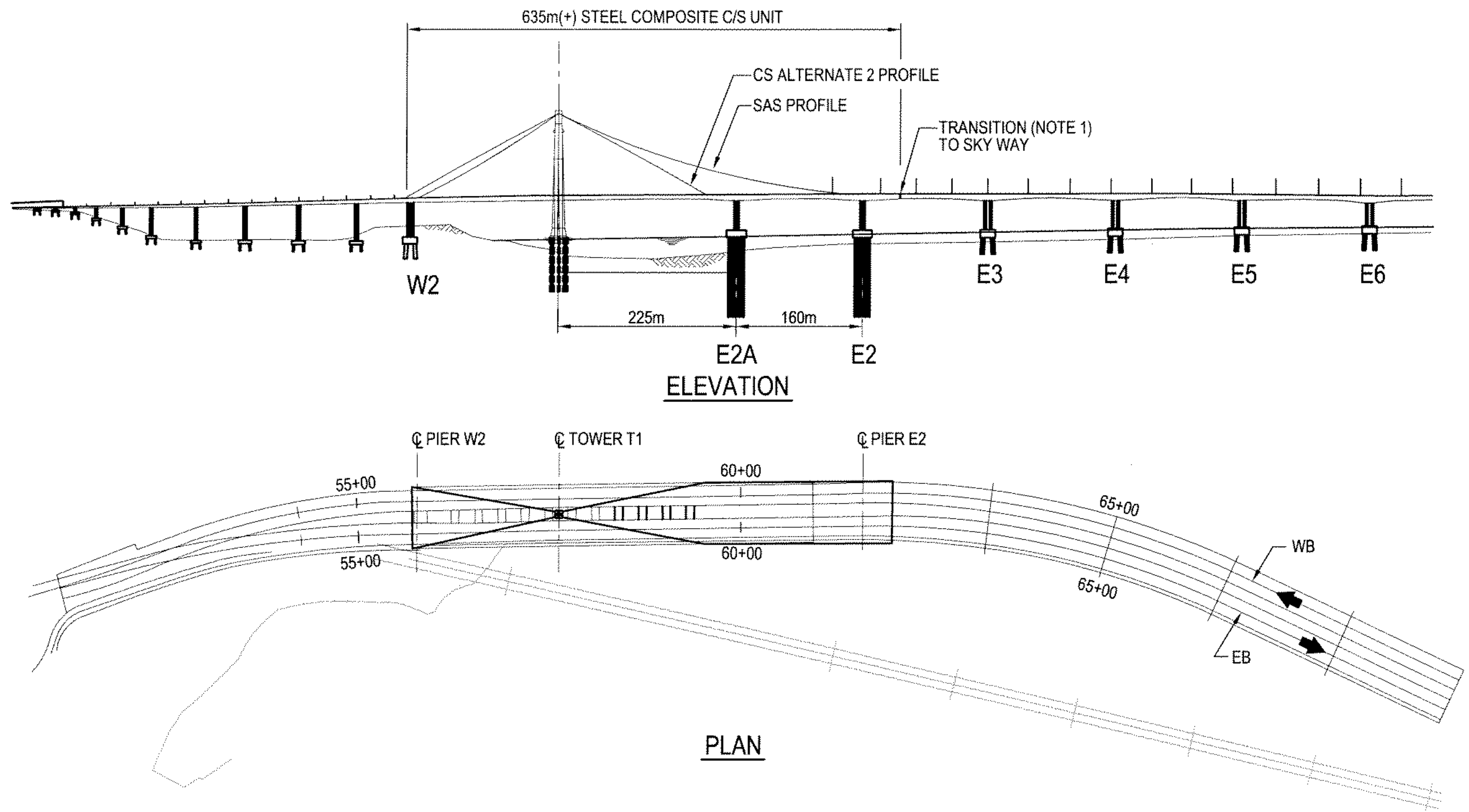
1. **Foundations:** For Option A, foundations T1 and E2 can be used as-is or could be substantially reduced by redesign. The size of Pier E2A would be very similar to the redesigned Pier E2. For Option B, foundation T1 can be used as is or reduced by redesign. Foundations E2 and E2A would be similar to that at E3.

2. **Seismic Performance:** It is expected that the seismic performance levels specified in the SAS design criteria can be met or exceeded for all of the elements examined. This includes meeting the strain levels with foundation elements, towers, piers, superstructure, shear link performance, and all other global elements that were the focus of this preliminary design development.

3. **Tower Design:** The concrete towers can be designed to meet the seismic performance requirements of the project using less than 4% rebar steel as required by ATC-32. Also, the limits on tower concrete and steel strains assumed for the present study show that the tower can be designed to a seismic performance standard far exceeding those adopted for the SAS tower design.

4. **Impacts to YBI and Skyway Designs:** The transition option A allows complete elimination of the design impacts to Skyway. The design impacts to YBI are minimal and can be readily incorporated in to the design. For transition option B, we believe the feasible solutions exist.

5. **Interface Forces & Movements:** It is expected that the existing hinge design and details can be used with little or no change.



NOTE:

1. SEE BRIDGE ELEVATION SHEETS 2 AND 3 FOR SKY WAY TRANSITION OPTIONS A AND B.

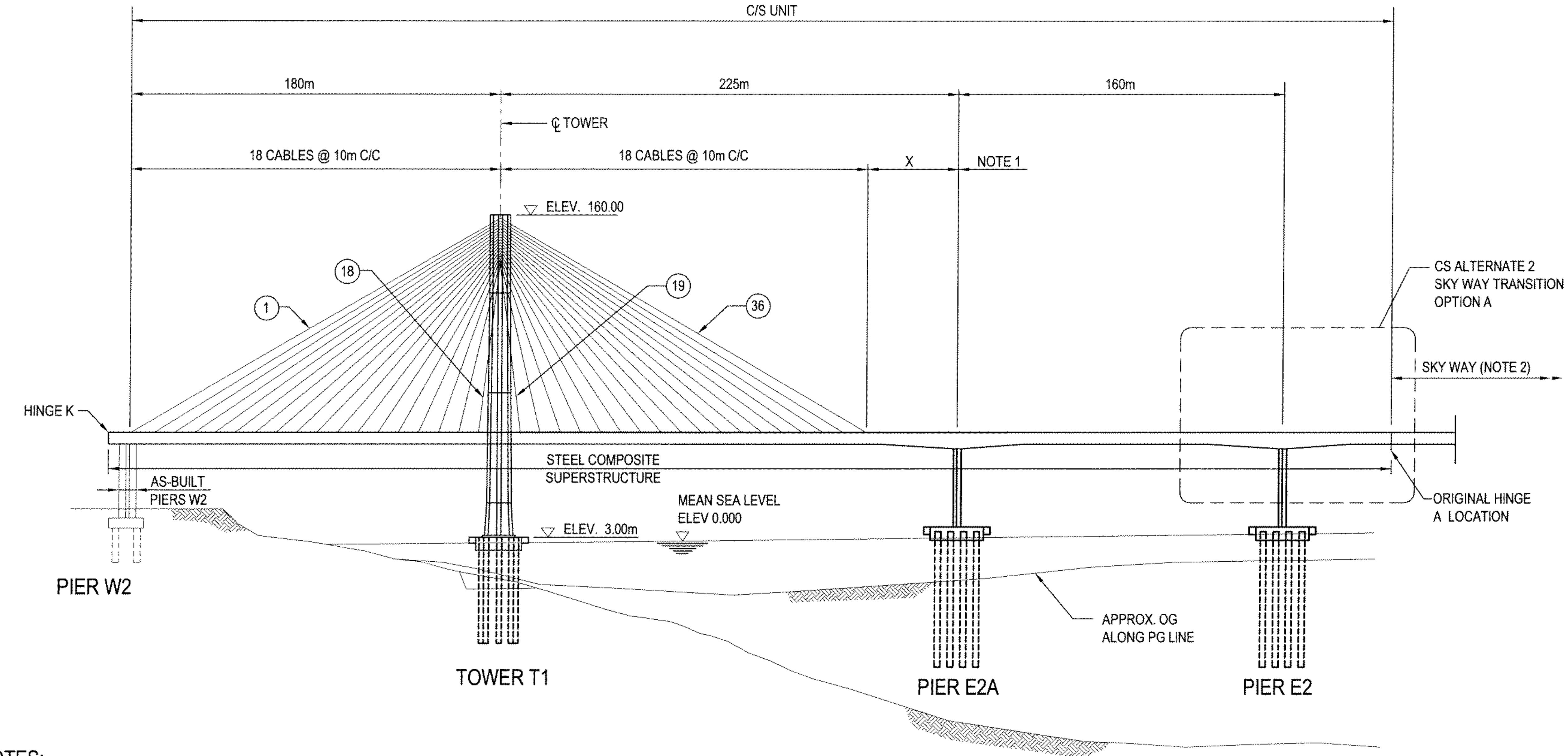
M:\39442-SFOBB\DWG\ALTERNATE 2\ALT2 GPE.DWG 11/17/04 16:25

Sheet 1 of 6

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 2: 180m-225m TWO SPAN LAYOUT
GENERAL PLAN & ELEVATION



NOTES:

1. THE NUMBER OF CABLES AND TOWER HEIGHT TO BE REFINED BASED ON THE OPTIMAL DISTANCE X (TO BE DETERMINED IN NEXT STAGE OF DEVELOPMENT)

2. HINGE LOCATION: PLACED TO MATCH EXISTING HINGE A LOCATION. CS SUPER STRUCTURE BEYOND PIER E2 TRANSIT TO MATCH EXISTING INTERFACE DETAILS.

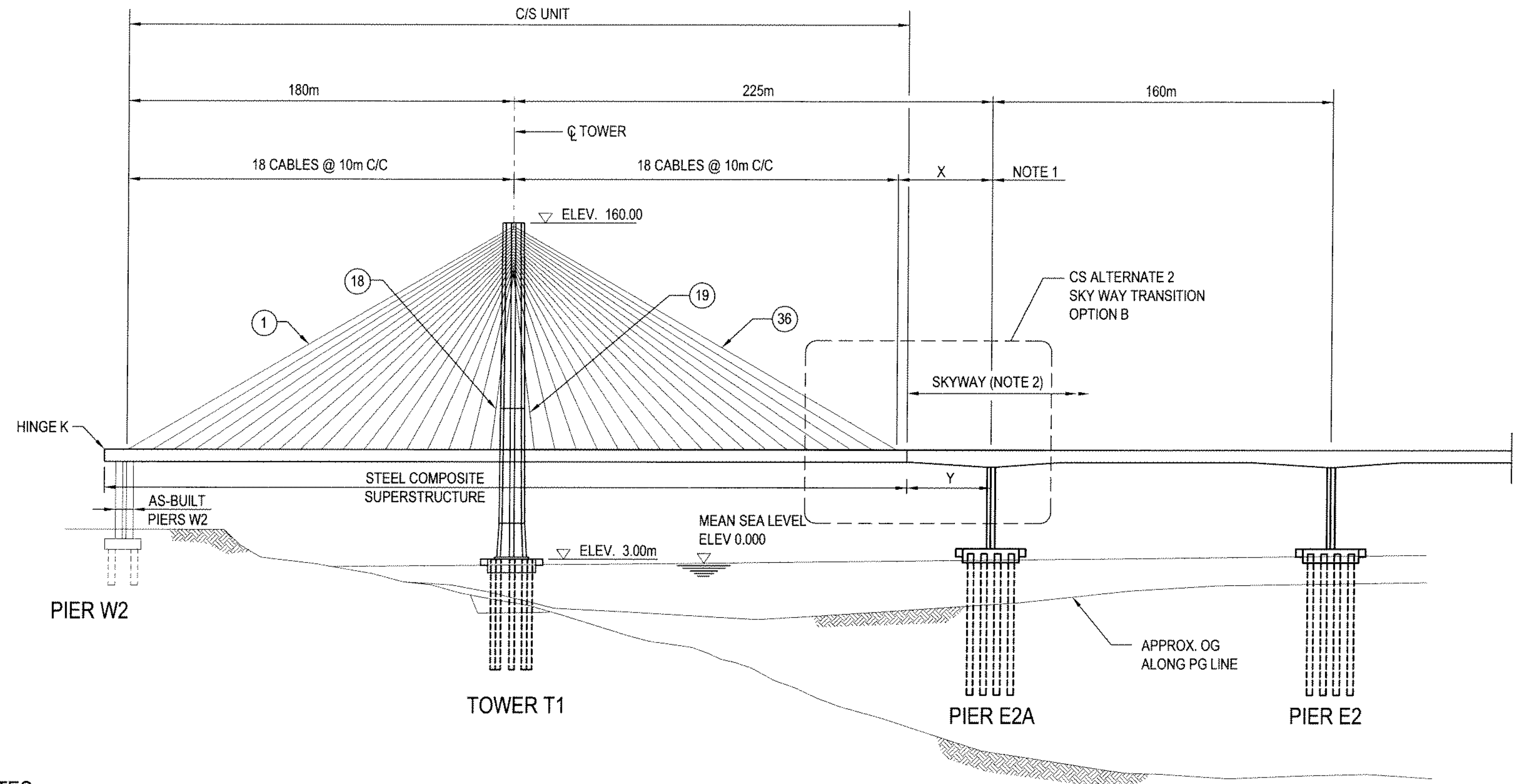
Sheet 2 of 6

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 2: 180m - 225m TWO SPAN LAYOUT
BRIDGE ELEVATION

M:\39442-SFOBB\DWG\ALTERNATE 2\ALT2 ELEVATIONA.DWG 11/17/04 16:24



M:\39442-SFOBB\DWG\ALTERNATE 2\ALT2 ELEVATIONB.DWG 11/17/04 16:25

NOTES:

1. THE NUMBER OF CABLES AND TOWER HEIGHT TO BE REFINED BASED ON THE OPTIMAL DISTANCE X (TO BE DETERMINED IN NEXT STAGE OF DEVELOPMENT)

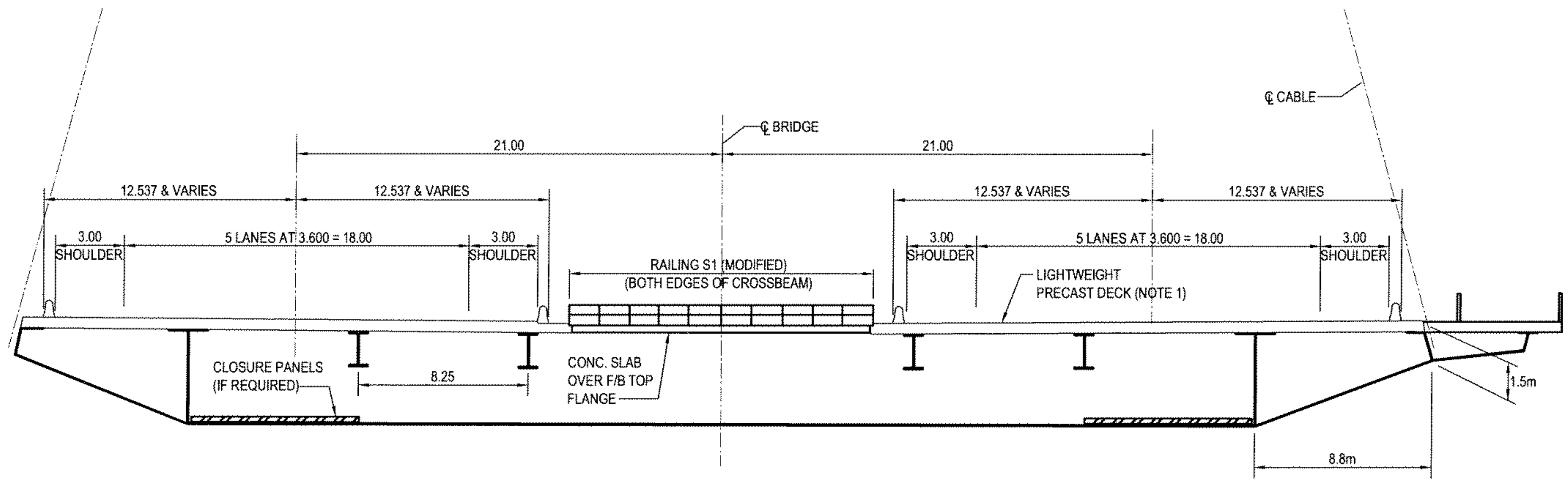
2. TRANSITION HINGE LOCATED ON WEST SIDE OF PIER E2A. DISTANCE Y TO BE REFINED TO SUIT DISTANCE X (NOTE 1) AND SKYWAY DESIGN OPTIMIZATION.

Sheet 3 of 6

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 2: 180m - 225m TWO SPAN LAYOUT
BRIDGE ELEVATION



STEEL COMPOSITE SUPERSTRUCTURE SECTION

(NOTE 1)

NOTE:

1. CONCRETE DECK ASSUMED FOR PRELIMINARY DESIGN (HEAVIEST OPTION). STEEL ORTHOTROPIC DECK CAN ALSO BE USED IN PLACE OF THE CONCRETE DECK. FOR THE CONCRETE DECK OPTION SHOWN, DECK SLAB DETAILS AND SECONDARY FLOOR FRAMING ARE NOT SHOWN.
2. SUPERSTRUCTURE CABLE ANCHORAGE LOCATIONS SHOWN ARE SCHEMATIC.

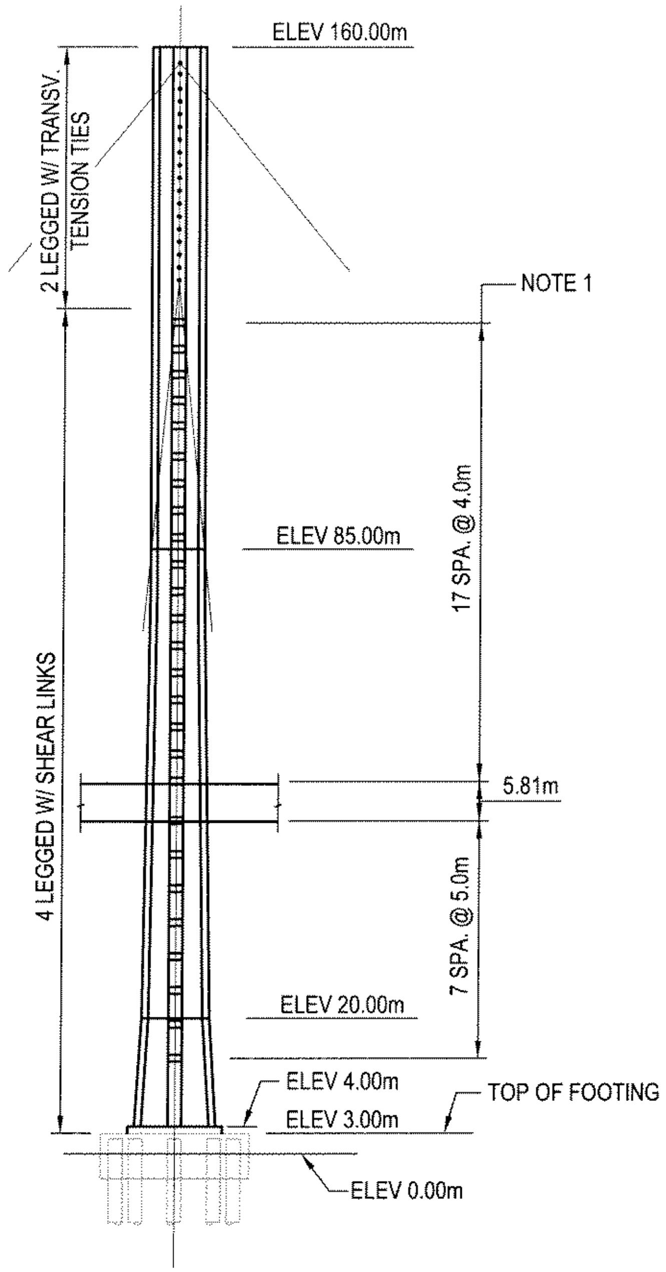
Sheet 4 of 6

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

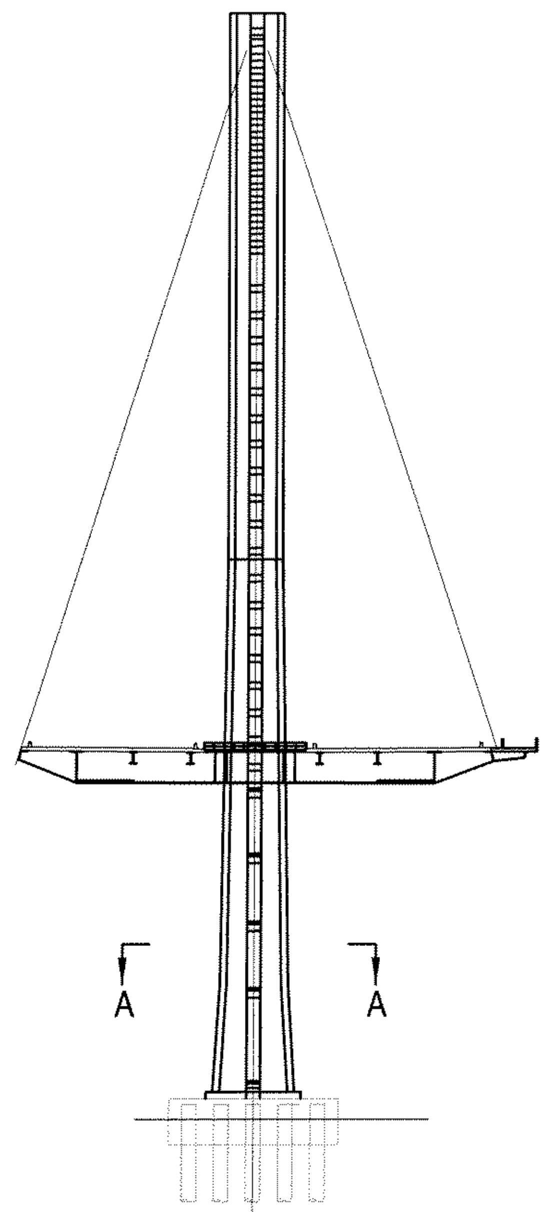
HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 2: 180m - 225m TWO SPAN LAYOUT
SUPERSTRUCTURE CROSS SECTION

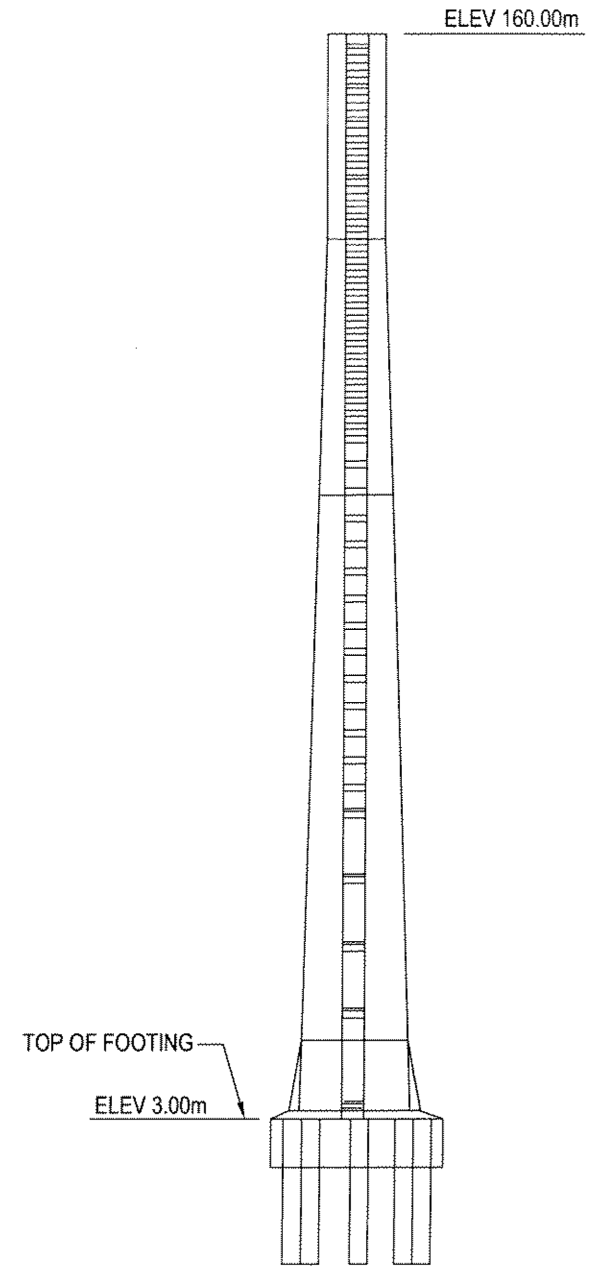
M:\39442-SFOBB\DWG\ALTERNATE 2\ALT2 SUPER SECTION.DWG 11/17/04 16:26



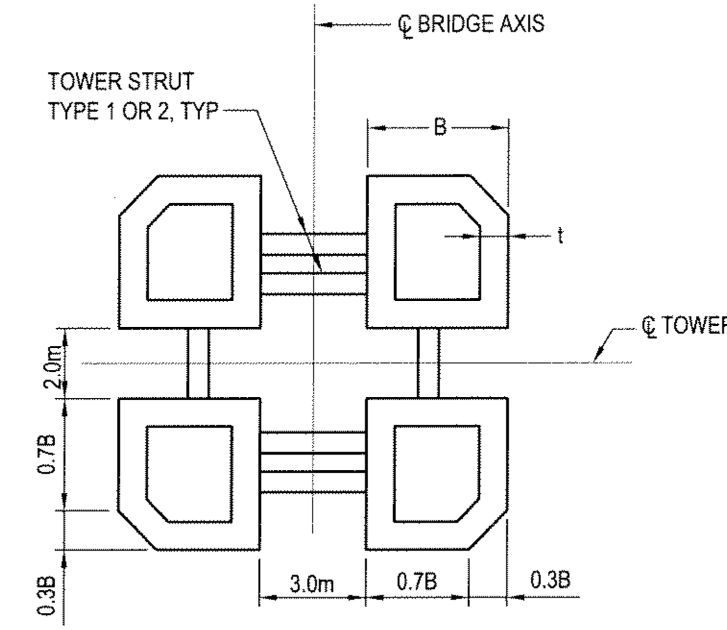
LONGITUDINAL ELEVATION
(LOOKING NORTH)
1:500



TRANSVERSE ELEVATION
(LOOKING EAST)
1:500



TRANSVERSE ELEVATION - SAS
1:500



TOWER SECTION A-A

TOWER SECTION TABLE		
Elev.	t	B
3.0m TO 20.0m	1.0m	Varies 5.0m TO 4.0m
20.0m TO 85.0m	0.8m	Varies 4.0m TO 3.0m
85.0m TO 160.0m	0.6m	3.0m

NOTE:

1. SHEAR LINKS SPACING IS SAME AS SAS.

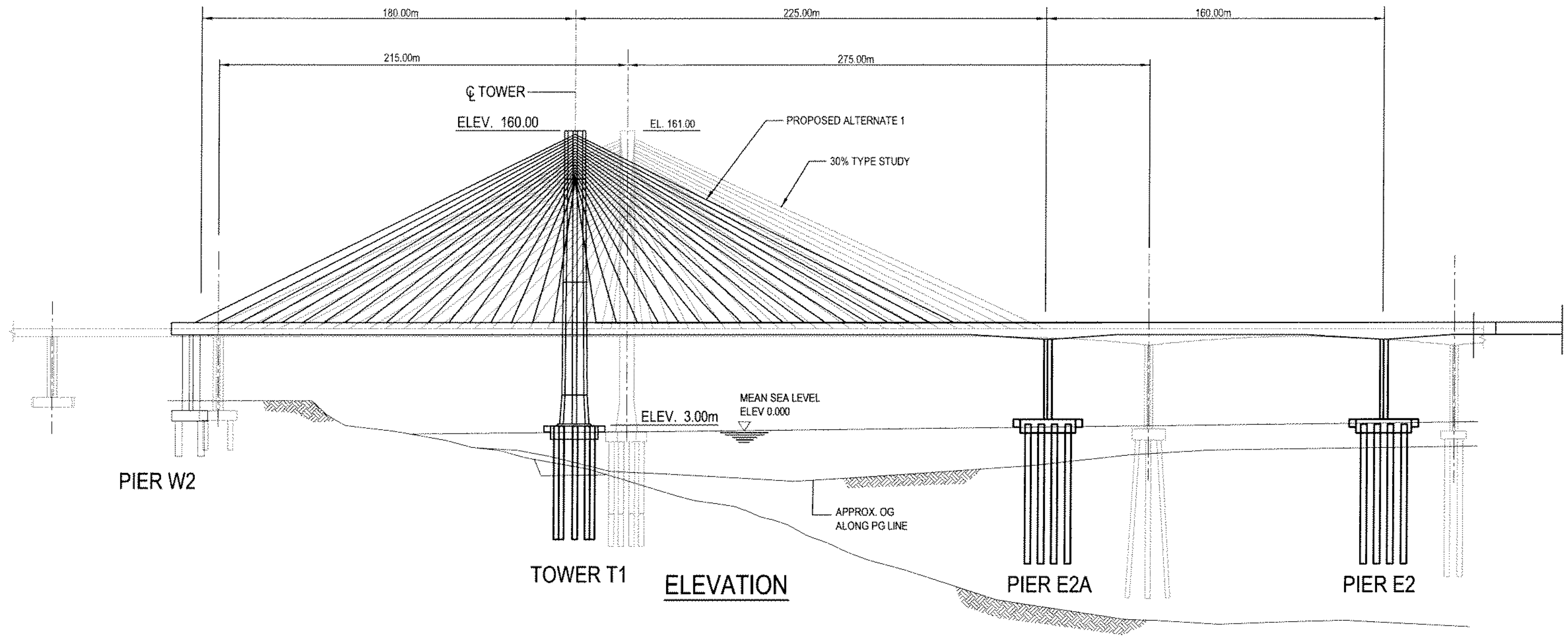
Sheet 5 of 6

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

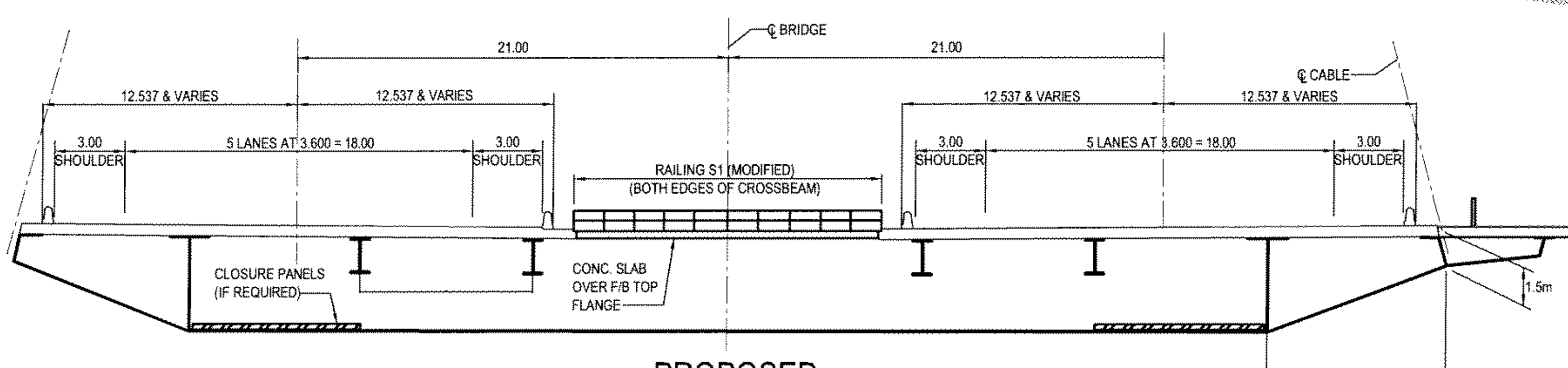
HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 2: 180m - 225m TWO SPAN LAYOUT
TOWER ELEVATIONS & SECTION

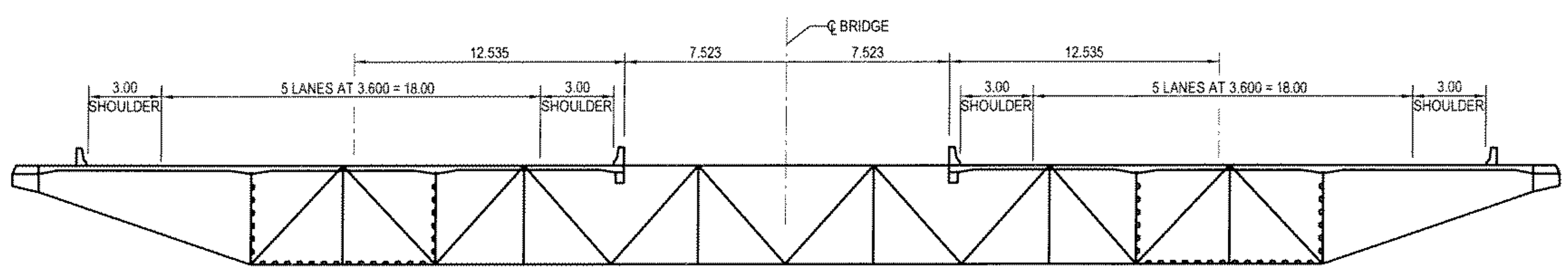
M:\39442-SFOBB\DWG\ALTERNATE 2\ALT2 TOWER.DWG 11/17/04 16:26



ELEVATION



PROPOSED



30% TYPE STUDY CROSS SECTIONS

M:\39442-SFOBB\DWG\ALTERNATE 2\ALT 2 EXISTELEV.DWG 11/17/04 16:27

Sheet 6 of 6

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 2: 180m - 225m TWO SPAN LAYOUT
COMPARISON WITH 30% TYPE STUDY

Cable-Stayed Alternate 3

6. CABLE-STAYED ALTERNATE 3 140M – 385M – 140M THREE SPAN LAYOUT

6.1 Description of Cable-Stayed Alternate 3 Structural Layout:

The preliminary structure layouts shown in Drawings 1 to 6 were developed following the process described previously in Section 2. The development assumptions and key features of CS Alternate 3 are as described in the following:

The deck weight assumed is the heaviest of the options previously listed in Table 3.1. The Cable-Stayed Alternate 3 was developed with only one transition option on the Skyway side. This transition occurs close to the beginning of the steel nose section[17], which will be eliminated with Cable-Stayed Alternate 3. Alternate 3 allows the selection of this transition location to be at an optimal location with respect to the Skyway design, so no major design changes to the skyway would be necessary. The hinge hardware, however, may require minor modifications to their mounting details (to the concrete section vs. previous steel section).

The two-tower system provides enhanced stability under seismic and other loading conditions compared to a single-tower system. This is due to the enhanced "frame" type action in a two-tower system when compared to typical "flagpole" type action in a single tower design. While not essential for its technical feasibility, in order to make the best use of the reduced foundation demands under the Alternate 3 configuration, foundations at T1 and E2 should be reconfigured. This would achieve the best overall performance of the structure and further optimize the overall cost and schedule. For the purpose of this report however, the analysis is based on using the T1 foundation as is, and the E2 foundation practically unchanged with the exception of a minor modification that would not require a major redesign effort.

The key technical challenges encountered in the development of Cable-Stayed Alternate 3 are:

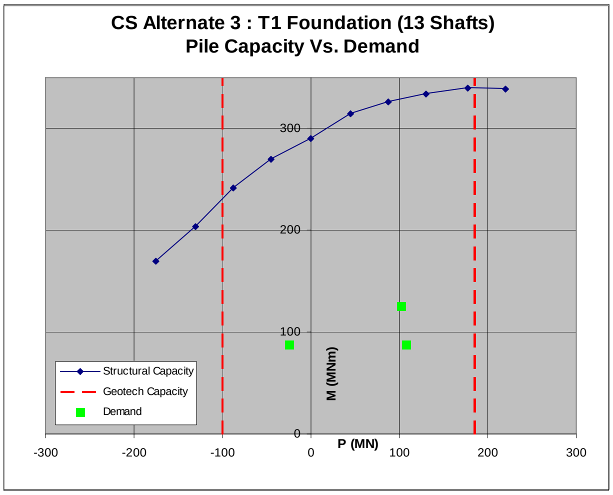
- A relatively significant amount of tension in the superstructure was predicted by the TY Lin's analysis. However, this significant tension was not found in HNTB's independent model. It is our opinion that this tension could be the result of a modeling issue that can be corrected through finer examination, or could be eliminated through better proportioning of the two foundations. However, as a conceptual solution, we have shown a joint at the middle of the main span until this issue is resolved.

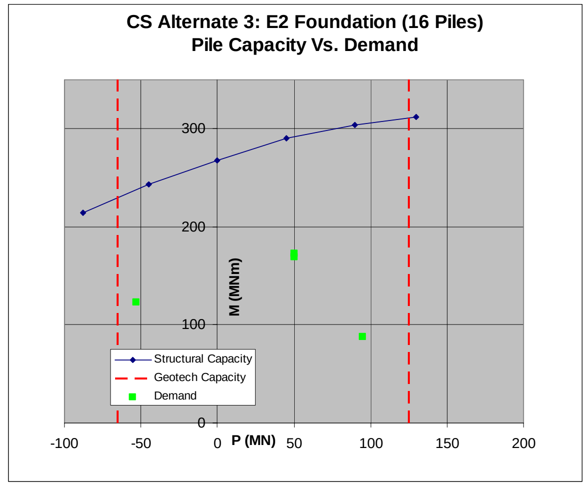
- As noted previously, while existing foundation designs can be used as-is, a better technical solution and higher level of economy can be realized through a foundation redesign where the foundation sizes are reduced to suit the bridge. It could be possible to do this by deleting some parts of the existing footings to minimize the design effort and the time.

[17] With the current SAS design, the segmental concrete typical Skyway superstructure is too heavy to be continued to Hinge A where it meets the SAS bridge. Thus, the skyway is transformed to a steel nose section to lighten up the cantilever.

6.2 Results of Analysis and Design Checks (for Layout Shown In Drawings)

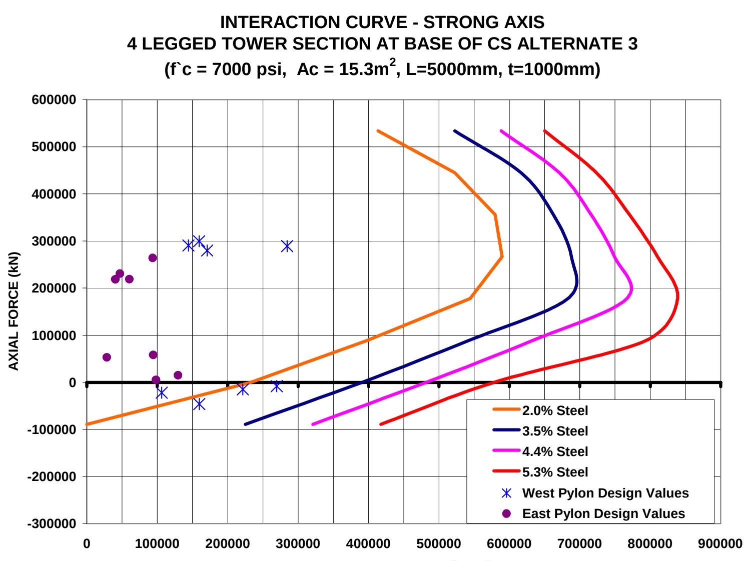
1. **Foundations:** The SAS foundations can be used as-is for Cable-Stayed Alternate 3. The following graphs show the demand plotted against the capacity for the drilled shafts at T1 and driven piles at E2. As shown in Drawing Sheet 5 of 6 for Alternate 3, the E2 foundation has been slightly modified by eliminating the foundation strap[18]. The pile capacities have been computed based on the similar criteria and design data as used in the SAS design.





The drilled shafts at T1 and piles at E2 have large amounts of additional capacity, and both foundations can be substantially reduced through proper redesign. The E2 piles could be battered similar to the skyway piles to make them more effective.

2. **Concrete Tower:** The following graph shows the tower base demand for the controlling seismic loads plotted against the capacity of the tower legs, based on 0.002 strain level in concrete and first yield of rebar obtained from Caltrans' X-Section Program.



The above plot indicates that the tower legs can meet the seismic demand under the very stringent criteria adopted for the check, and have excess capacity allowing for further design optimization

[18] To bring the two foundation units close together to support the tower stem

(and reduced seismic demands). This verifies that the concrete towers can be designed to meet or exceed the SAS/SFOBB seismic design and performance criteria. The difference in seismic moments between the west and east pylons show that the global structural system can be optimized further.

3. **Tower Shear Links:** The same shear link properties and shear link placement as the SAS was assumed for Cable-Stayed Alternate 3 until the cable anchorage area was reached. The analysis results show that the performance of the shear links are within the SAS seismic design criteria except for the longitudinal links in the West Pylon. This demonstrates that some additional refinements in the proportioning between the towers and the two foundations are required for this option in order to meet the desired seismic design criteria.

Shear Link Orientation	Shear Link Plastic Rotations (Radians)	
	CABLE-STAYED Alternate 3	Limiting Rotation per SAS Design Criteria
Longitudinal	0.030 East Pylon 0.100 West Pylon	0.08
Transverse	0.025	

4. **W2 and E2 Pier Columns:** The seismic performance of the W2 and E2 pier columns for the cable-stayed alternatives can be verified relatively quickly by comparing the moment demand for the cable-stayed with those for the SAS design. This provides a firm verification that the pier columns can provide the same level of seismic performance as incorporated into the SAS design. The axial loads on the pier columns can easily be adjusted to suit by refining cable forces, providing post tensioning, or some measure of both as needed. The following tables compare the maximum demand per pier column at Pier W2 and per pier column at Pier E2 relative to the corresponding SAS design demands.

Pier W2: Maximum design demand per pier column

	Axial MN	L-Mom MNm	T-Mom MNm
SAS	170 / -100	300	230
Cable-Stayed Alternate 3	139 / -125	309	263

The demands on pier columns at W2 for Cable-Stayed Alternate 3 are practically the same as SAS levels and are within the capacity of W2 pier columns. It is expected that further design refinements can substantially improve the overall system response to seismic loads.

5. **Interface Forces:** The governing forces at the Skyway and YBI interfaces are listed below:

Interface Forces and Movements - Cable-Stayed Alternate 3				
		Forces		Movement
		Transverse Shear (MN)	Vertical Shear (MN)	Longitudinal Displacement (mm)
YBI	Cable-Stayed Alternate 3	25	41	759
(Hinge K)	SAS	16	74	1285
Skyway	Cable-Stayed Alternate 3	102	19	1199
(Hinge A)	SAS	16	32	1170

The table shows that interface forces and movements for the CS Alternate 3 are acceptable except the transverse shear at the Skyway interface. This needs some further study to determine the cause of this and to provide a solution. If necessary, the pipe beams at the interface may have to be redesigned.

6. **Global:** Based on the results for Alternate 1, the superstructure stresses should not be a problem. Additionally, it is expected that further design developments involving global optimization of the structure layout can be used to solve the few remaining issues.

6.3 Conclusions of the Technical Analysis of Cable Stayed Alternative 3

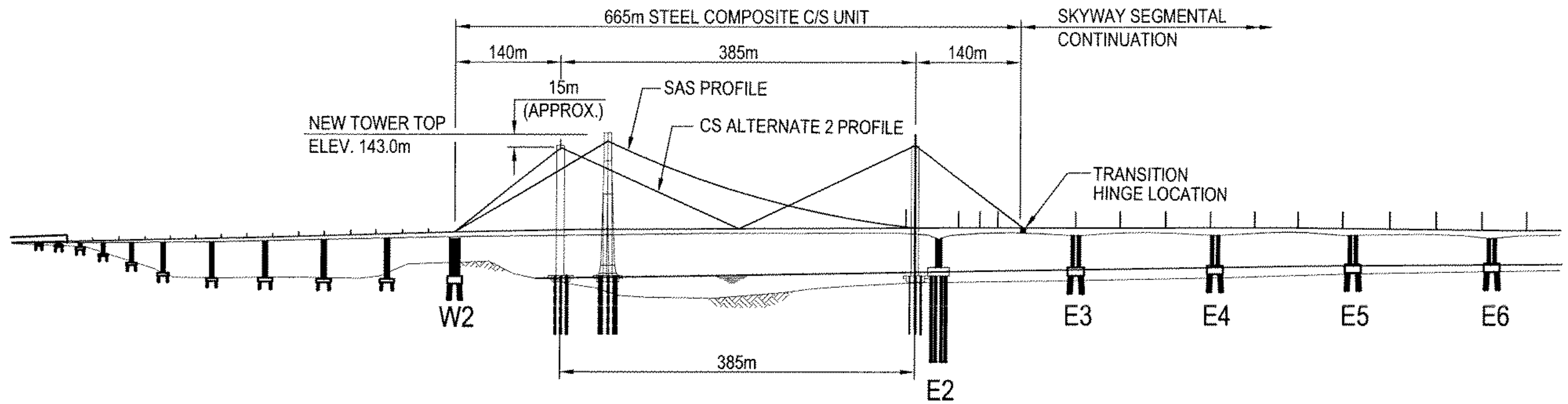
1. **General:** The analysis is based on conservative assumptions with respect to key elements such as the superstructure weight, tower weight, and tower stiffness. The demands for the foundations and towers during the next stage of design development are expected to be lower than those predicted at this stage. This alternate requires some further design refinements to resolve a couple of remaining issues. However, the key issues such as the foundation sizes and the concrete tower performance have been confirmed.

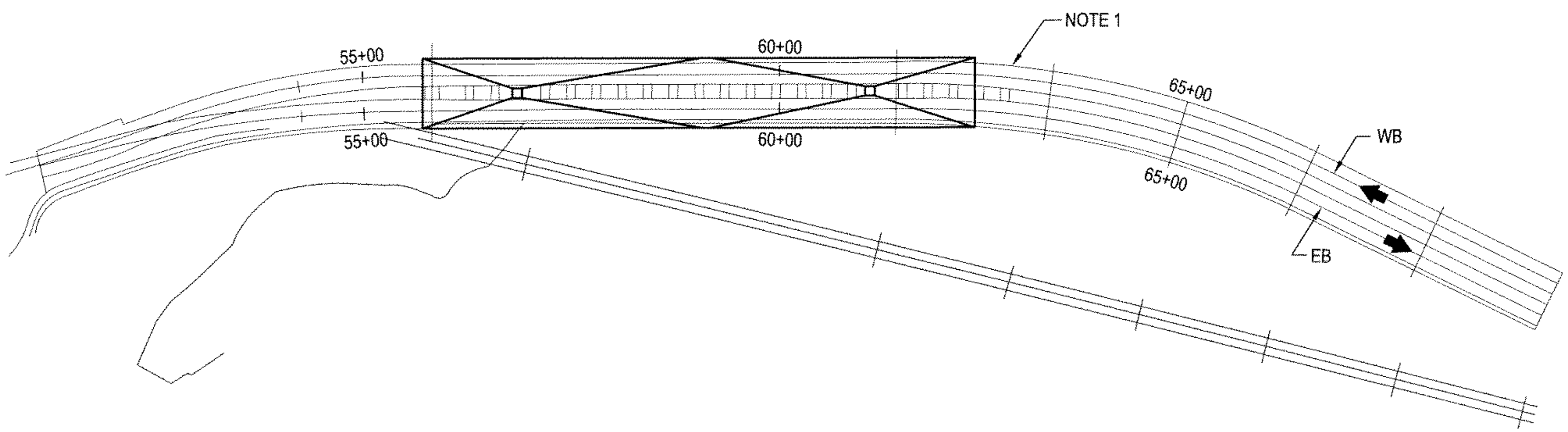
2. **Foundations:** The analysis shows that the existing T1 and E2 foundations can be used as-is for Alternate 3. However, to achieve the best bridge layout it is our opinion that the tower foundations must be properly redesigned to make them more proportional to the structure.

3. **Seismic Performance:** Seismic performance levels specified in the SAS design criteria can be met for the elements examined. This includes meeting the strain levels with foundation elements, towers, piers, superstructure, shear links (except west tower longitudinal), and all other global elements that were the focus of this preliminary design development. Further design refinements are needed to resolve the remaining issues.

4. **Tower Design:** The concrete towers can be designed to meet the seismic performance requirements of the project using less than 4% rebar steel as required by ATC-32. Also, the limits on tower concrete and steel strains assumed for the present study show that the tower can be designed to a seismic performance standard far exceeding those adopted for the SAS tower design.

5. **Impacts to YBI and Skyway Designs:** The transition design impacts to the Skyway are not major. The only issue needing resolution is the large magnitude transverse shear predicted by TY Lin's analysis at the hinge location. The design impacts to YBI are minimal and can be readily incorporated into the design.





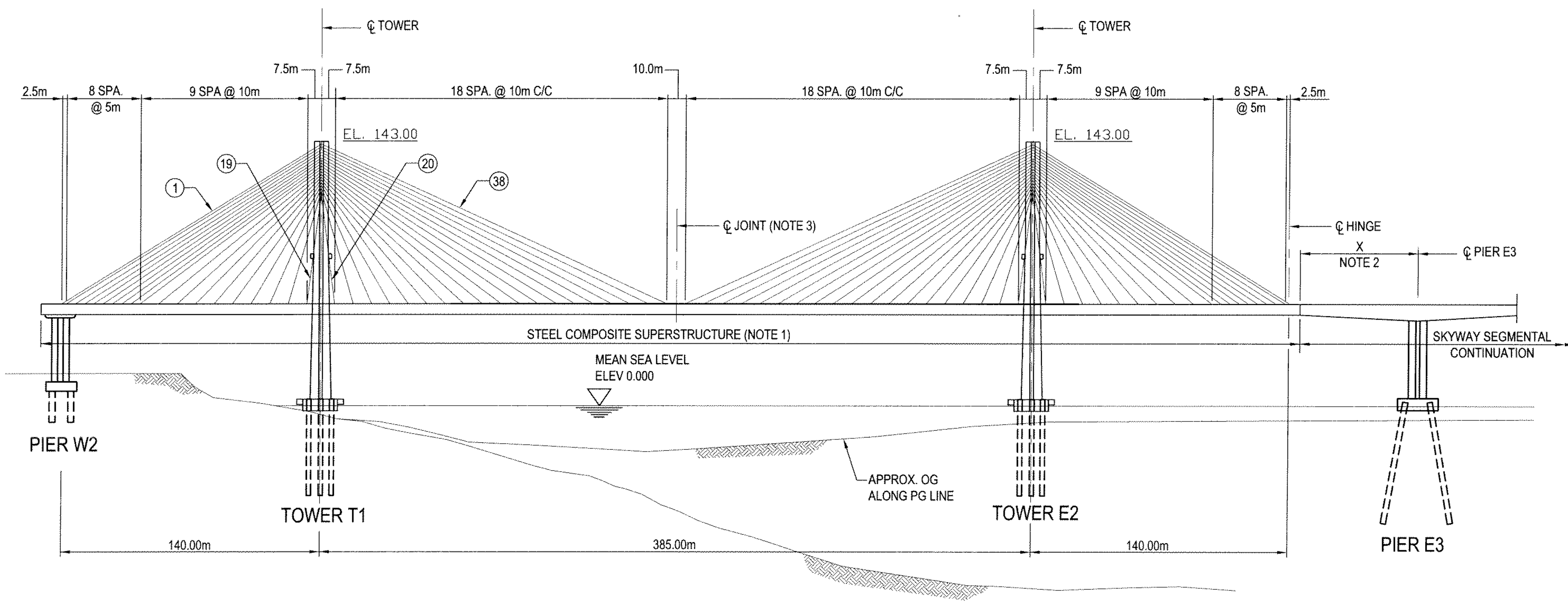
NOTES
1. SMALL CURVATURE MODIFICATION PREFERABLE, NOT ESSENTIAL

Sheet 1 of 6

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 3: 140m - 385m - 140m 3 SPAN LAYOUT
GENERAL PLAN & ELEVATION



NOTES

1. SUPERSTRUCTURE SIMILAR TO ALTERNATE 1, STEEL COMPOSITE SECTION.
2. DISTANCE X TO BE REFINED IN NEXT STAGE OF DESIGN DEVELOPMENT INDICATED AS A CONTINGENCY ONLY.
3. IT IS EXPECTED THAT FURTHER DESIGN DEVELOPMENT MAY ELIMINATE NEED FOR JOINT AT THIS LOCATION.

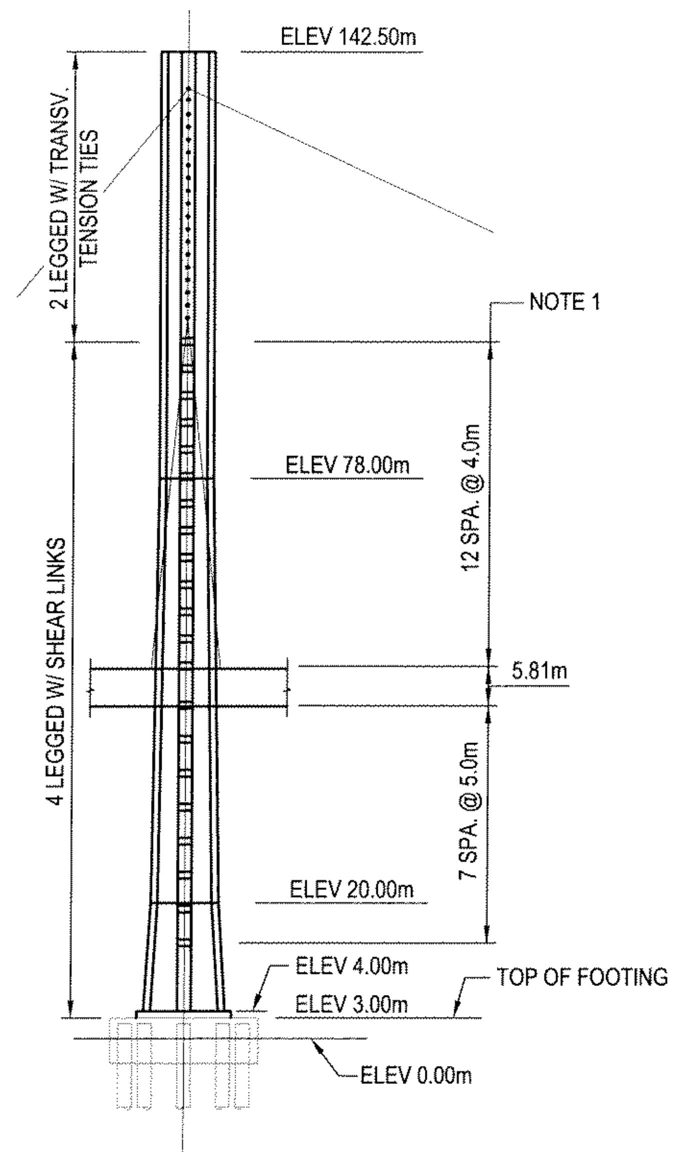
M:\39442-SFOBB\DWG\ALTERNATE 3\3 SPAN ELEVATION.DWG 11/17/04 16:30

Sheet 2 of 6

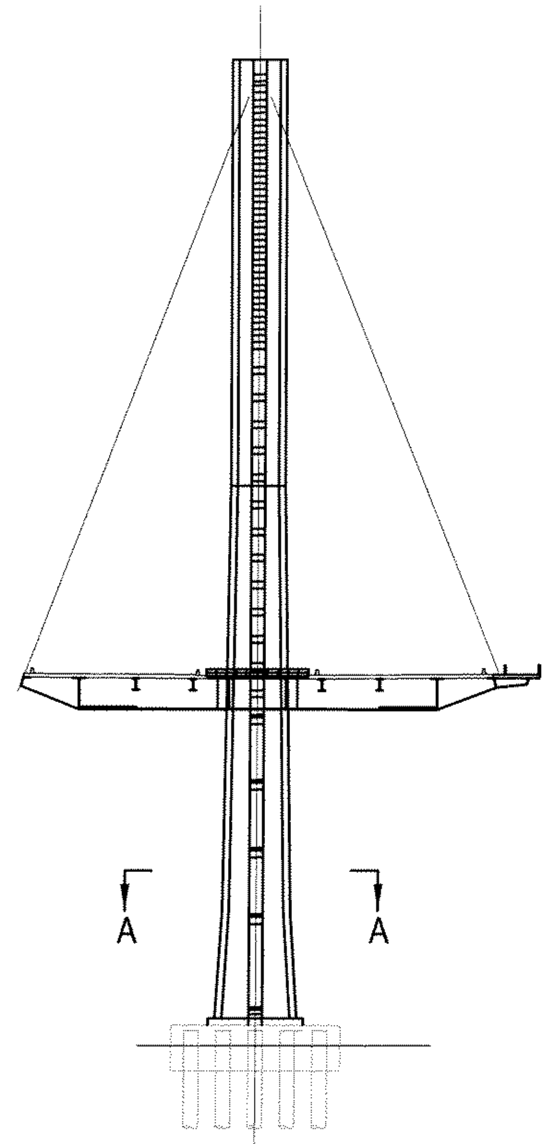
SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

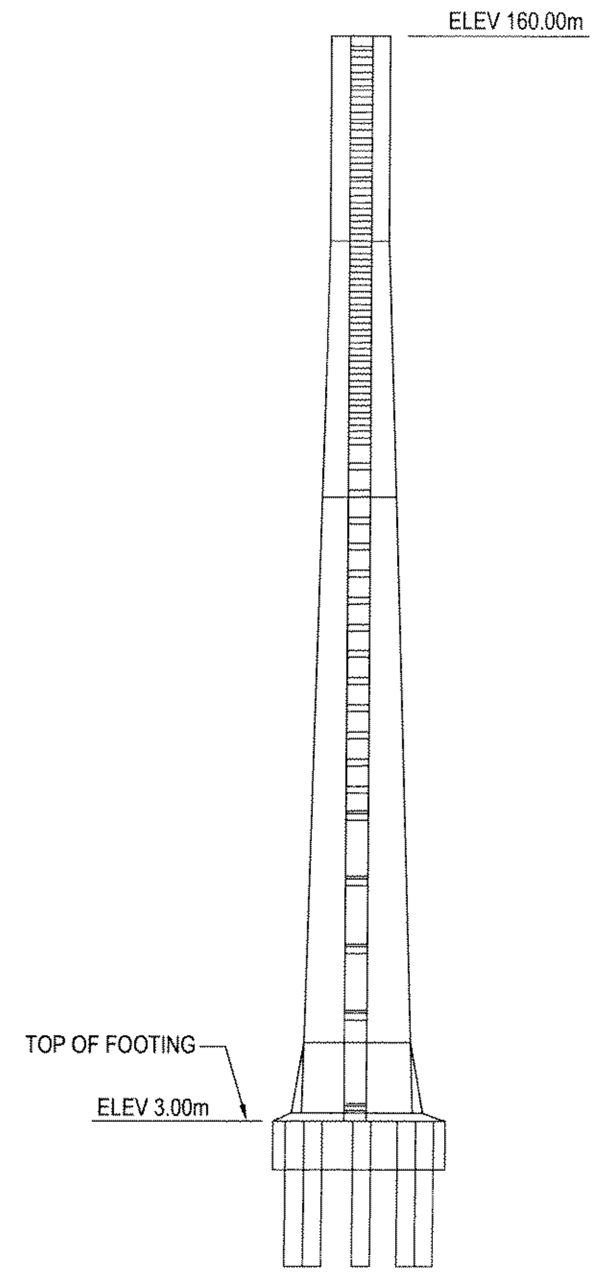
CS ALTERNATE 3: 140m -385m -140m 3 SPAN LAYOUT
BRIDGE ELEVATION



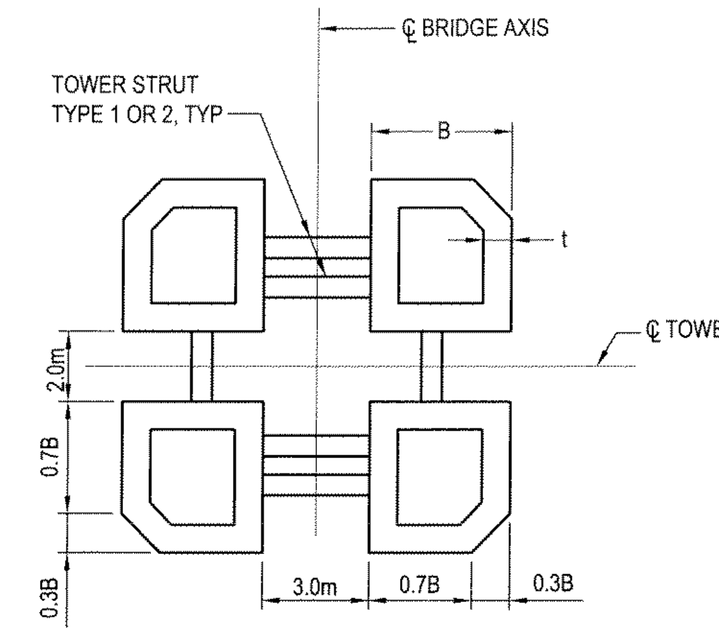
LONGITUDINAL ELEVATION
(LOOKING NORTH)
1:500



TRANSVERSE ELEVATION
(LOOKING EAST)
1:500



TRANSVERSE ELEVATION - SAS
1:500



TOWER SECTION A-A

TOWER SECTION TABLE		
Elev.	t	B
3.0m TO 20.0m	1.0m	Varies 5.0m TO 4.0m
20.0m TO 78.0m	0.8m	Varies 4.0m TO 3.0m
78.0m TO 142.5m	0.6m	3.0m

NOTE:

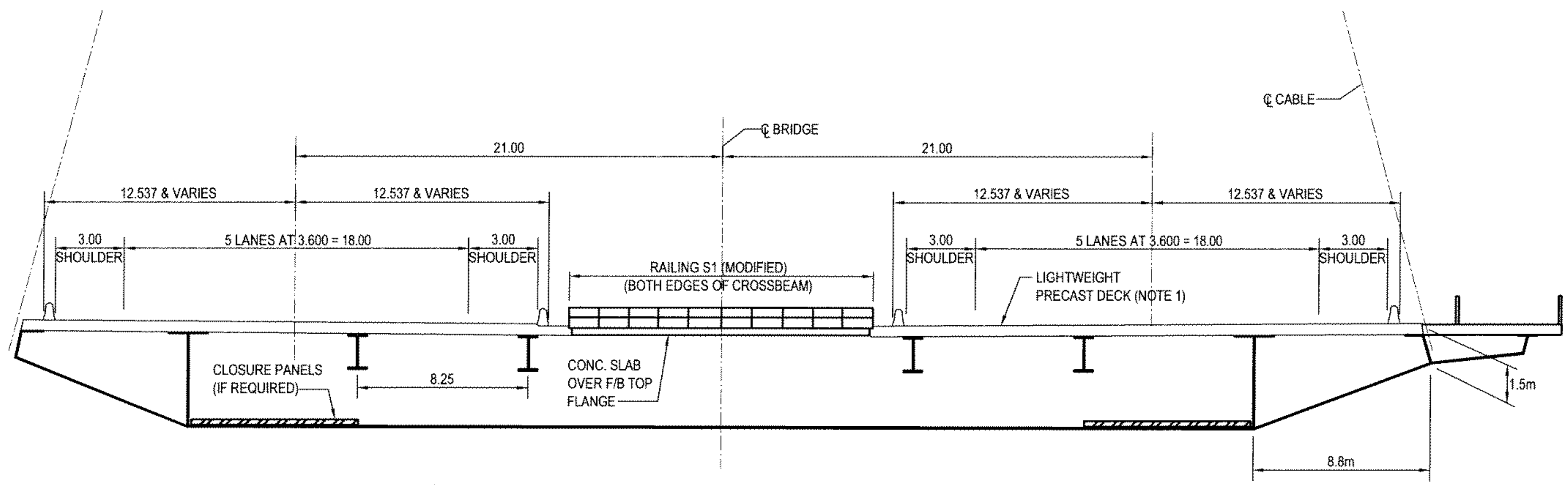
1. SHEAR LINKS SPACING IS SAME AS SAS.

Sheet 3 of 6

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 3: 140m -385m -140m 3 SPAN LAYOUT
TOWER ELEVATIONS & SECTION



STEEL COMPOSITE SUPERSTRUCTURE SECTION

(NOTE 1)

NOTE:

1. CONCRETE DECK ASSUMED FOR PRELIMINARY DESIGN (HEAVIEST OPTION). STEEL ORTHOTROPIC DECK CAN ALSO BE USED IN PLACE OF THE CONCRETE DECK. FOR THE CONCRETE DECK OPTION SHOWN, DECK SLAB DETAILS AND SECONDARY FLOOR FRAMING ARE NOT SHOWN.
2. SUPERSTRUCTURE CABLE ANCHORAGE LOCATIONS SHOWN ARE SCHEMATIC.

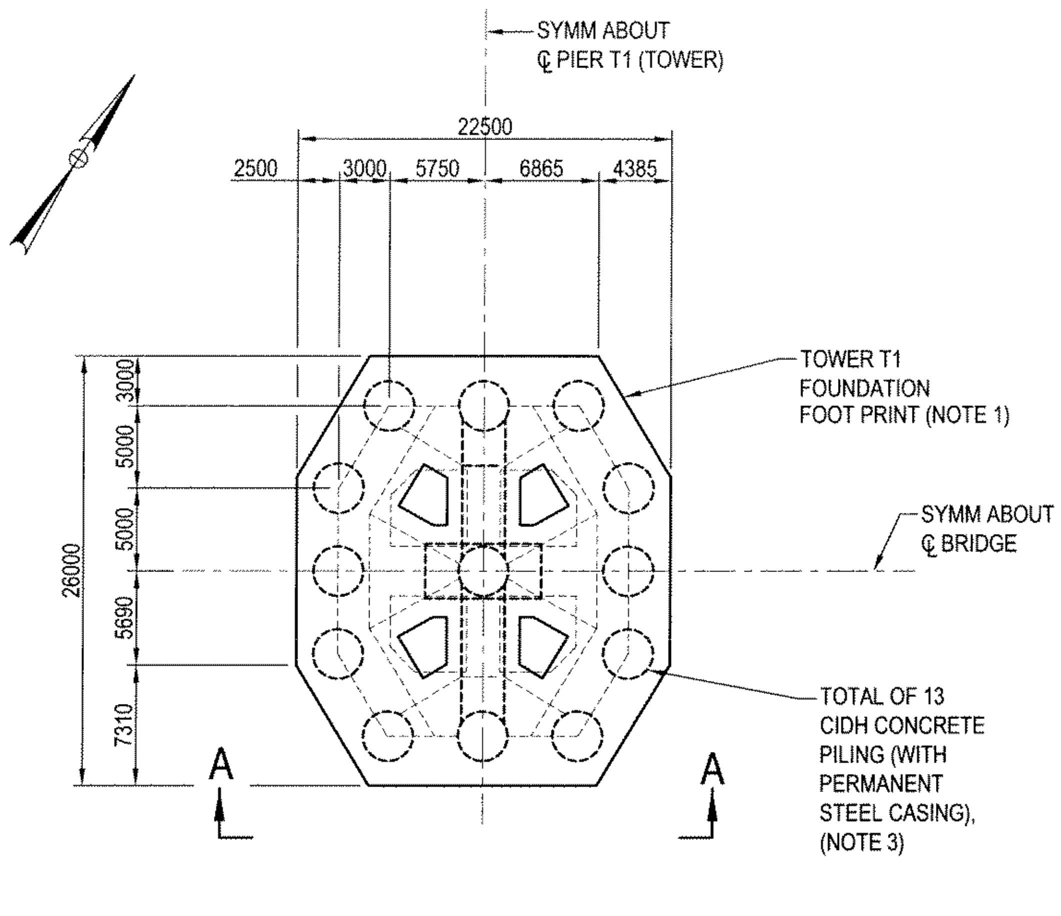
Sheet 4 of 6

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

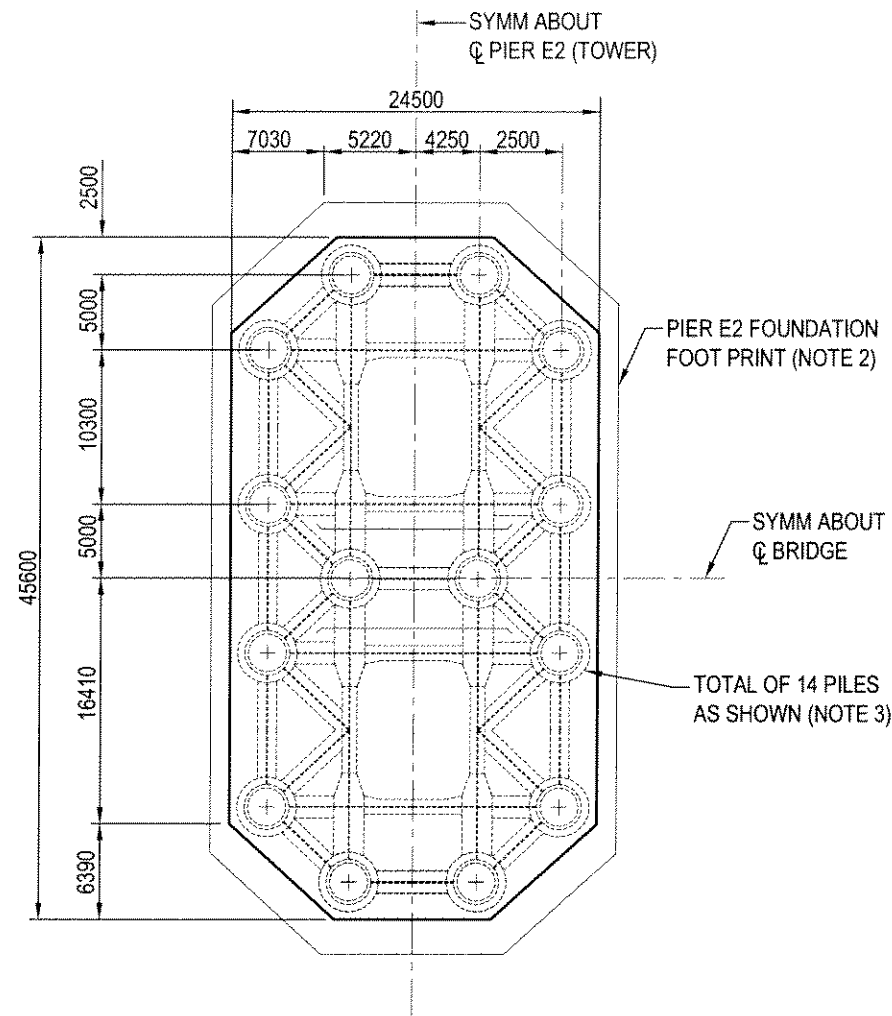
HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 3: 140m - 385m - 140m 3 SPAN LAYOUT
SUPERSTRUCTURE CROSS SECTION

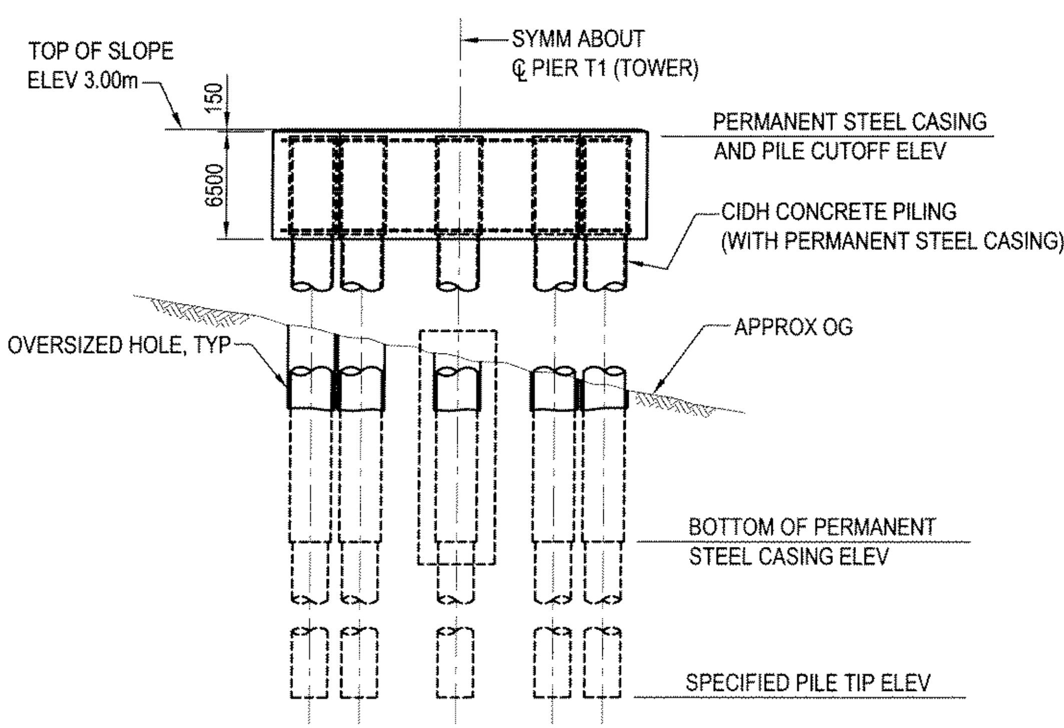
M:\39442-SFOBB\DWG\ALTERNATE 3\3 SPAN SUPER SECTION.DWG 11/17/04 16:31



PLAN T1



PLAN E2



ELEVATION A-A

NOTES:

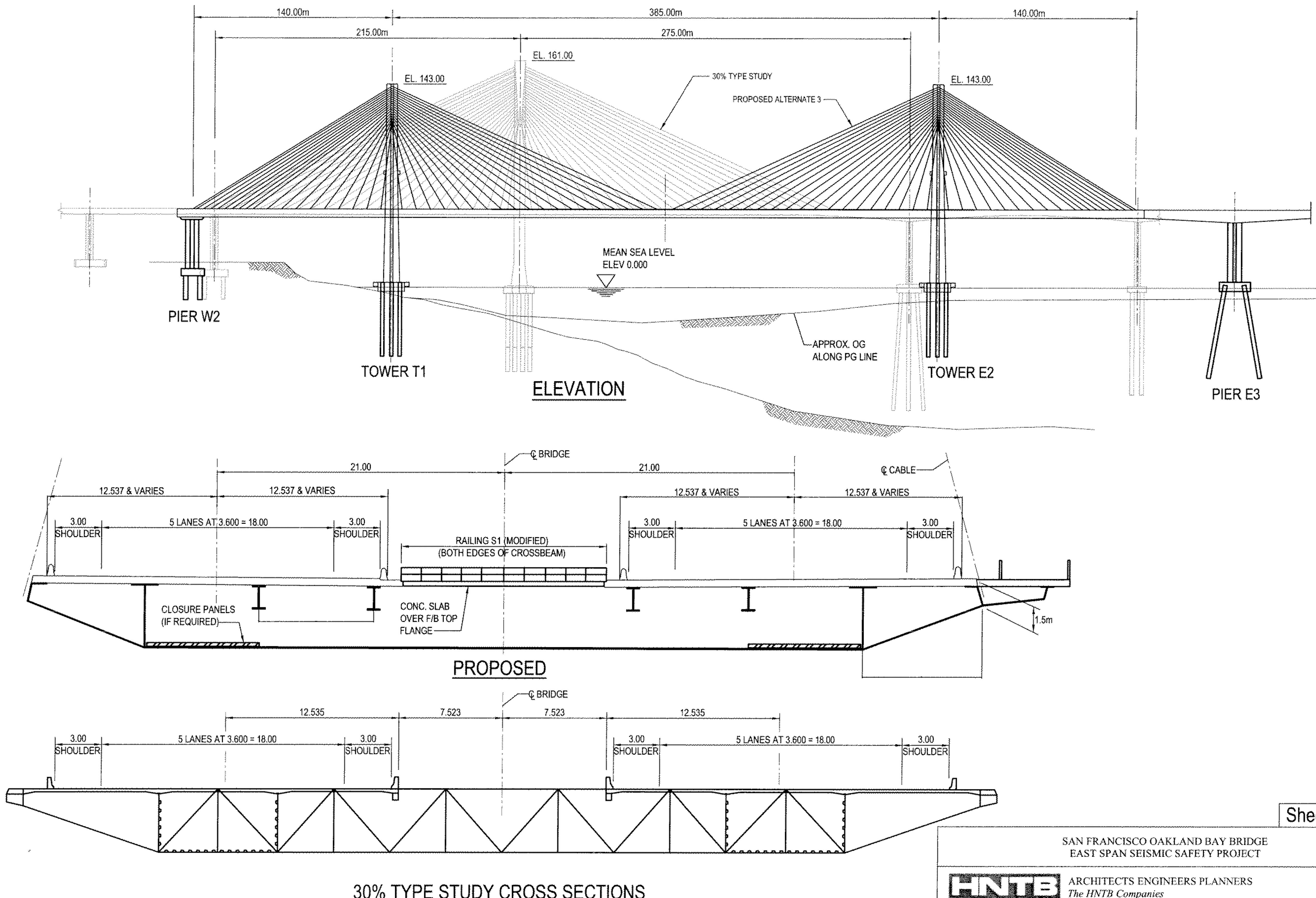
1. T1 FOUNDATION FOOT PRINT CAN REMAIN UNCHANGED AS SAS OR REDUCED SUBSTANTIALLY BY RE-DESIGN.
2. E2 FOUNDATION FOOT PRINT CAN BE REDUCED AS SHOWN OR REDUCED FURTHER BY RE-DESIGN.
3. NUMBER OF PILES UNCHANGED AT T1 (13) AND POSSIBLY REDUCE E2 BY TWO PILES AS SHOWN (14). THE NUMBER OF PILES CAN BE REDUCED SUBSTANTIALLY THROUGH RE-DESIGN.

Sheet 5 of 6

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 3: 140m -385m -140m 3 SPAN LAYOUT
T1 & E2 TOWER FOUNDATIONS



M:\39442-SFOBB\DWG\ALTERNATE 3\EXISTELEV2.DWG 11/17/04 16:33

Sheet 6 of 6

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

HNTB ARCHITECTS ENGINEERS PLANNERS
The HNTB Companies

CS ALTERNATE 1: 180m-385m 3 SPAN LAYOUT COMPARISON WITH TYPE STUDY

Schedules

7. SCHEDULES

The schedules attached at the end of this section present design and construction schedules for the following cable-stayed alternatives:

1. Alternate 1 – Scenario I, T1 and E2 Foundations as Designed
2. Alternate 1 – Scenario II, T1 and E2 Foundations Redesigned
3. Alternate 3 – Scenario I, T1 and E2 Foundations as Designed
4. Alternate 3 – Scenario II, T1 and E2 Foundations Redesigned

In addition, at the request of Caltrans, the IRT prepared a schedule for a skyway alternative.

The following discussion provides a basis for the development of the schedules:

7.1 Contracting for Architectural & Engineering Services

The schedules all have a start date of March, 2005, providing an expedited two-month period (January and February 2005) for selection, negotiation, and contract signing (an initial NTP may be necessary) of a Design Consultant for the redesign.

7.2 Environmental Schedule

For all alternates, we have assumed a 9-month environmental process, since the foundation sizes are the same or smaller (shifted 40 meters for Alternate 3). See Section 10 – Environmental Review by John Hesler.

7.3 Design Schedule

The design schedule essentially assumes the following design phase periods corresponding to separate bid packages for the foundations and the superstructure similar to SAS:

Alternate	Foundation Design	Superstructure Design
Alt. 1 Scenario I	6 mo.	18 mo. + 6 mo.
Alt. 1 Scenario II	12 mo	18 mo. + 6 mo.
Alt. 3 Scenario I	6 mo.	18 mo. + 6 mo.
Alt. 3 Scenario II	12 mo.	18 mo. + 6 mo.

For Alternate 1 Scenario II and Alternate 3 Scenario II, an additional 6-month foundation design time is assumed in order to redesign the foundation frames. As mentioned previously, based on the work to date, the IRT does not believe it will be necessary to add additional shafts and redesign the frame in the case of Alternate 1 Scenario II. The schedule also assumes that an 8-month work delay can be negotiated with Kiewit, the E2/T1 contractor, so that sufficient design work can be completed on the foundations to confirm the suitability of the existing foundation design or provide a redesign.

For Alternate 3, there is also 6-month of time allocated for geotechnical explorations due to the foundations being 40m away from where they are located with respect to the SAS design. However, the geotechnical data available should be reviewed to see if these additional explorations are necessary.

In arriving at the design schedule, it is important to note the design work that has already been completed. The following table provides a summary of design work already completed that can be incorporated into the final design of the cable-stayed bridge.

Design Element	Status	Remark
Geotechnical Information	Complete	Possible need for additional borings for Alternate 3
Ground Motions	Complete	
Adina Model (Foundations)	Complete	New Bridge Model
Preliminary CS Layouts	Complete	
T-1 Drilled Shafts	Complete	
E-2 Driven Piles	Complete	Redesign for Alternative 3
T-1Footing Frame	Complete	Possible Strengthening
E-2 Footing Frame	Complete	
Link Behavior	Established	
Bikeway	Complete	Redesign Connection
Hinges	Complete	Connection to Cable Stayed Bridge to be designed
W-2 Piers	Complete	Redesign Cap
E-2 Piers	Complete	Redesign Cap
Specifications	90% Complete – Foundations 50 – 75% Complete – Super-structure	
Miscellaneous	Complete	Minor Revisions

The superstructure schedule assumes an18-month duration to bid plans and advertisement. The bid plans would be fully detailed to Caltrans standards for all elements except the superstructure. Sufficient information would be provided, including quantities for which the contractor can submit a bid. Final details such as rebar details, weld details, splices, connections, etc., would be provided to the contractor within 6 months after the advertisement for bid (about two months after contract award. This process is familiar to Caltrans and referred to as contract sequencing. Based on familiarity with other similar projects, this 24-month period should be more than ample time, especially considering the significant amount of work already accomplished as mentioned previously.

During the design phase, review meetings with Caltrans and the Seismic Safety Panel are assumed at start of Preliminary Design and at 25%, 50%, 75%, and 100% design levels. Specifications can easily be accomplished between the 50% and 100% stages, since most of the specifications can be reused from the SAS, Skyway, W2, and E2/T1 contracts.

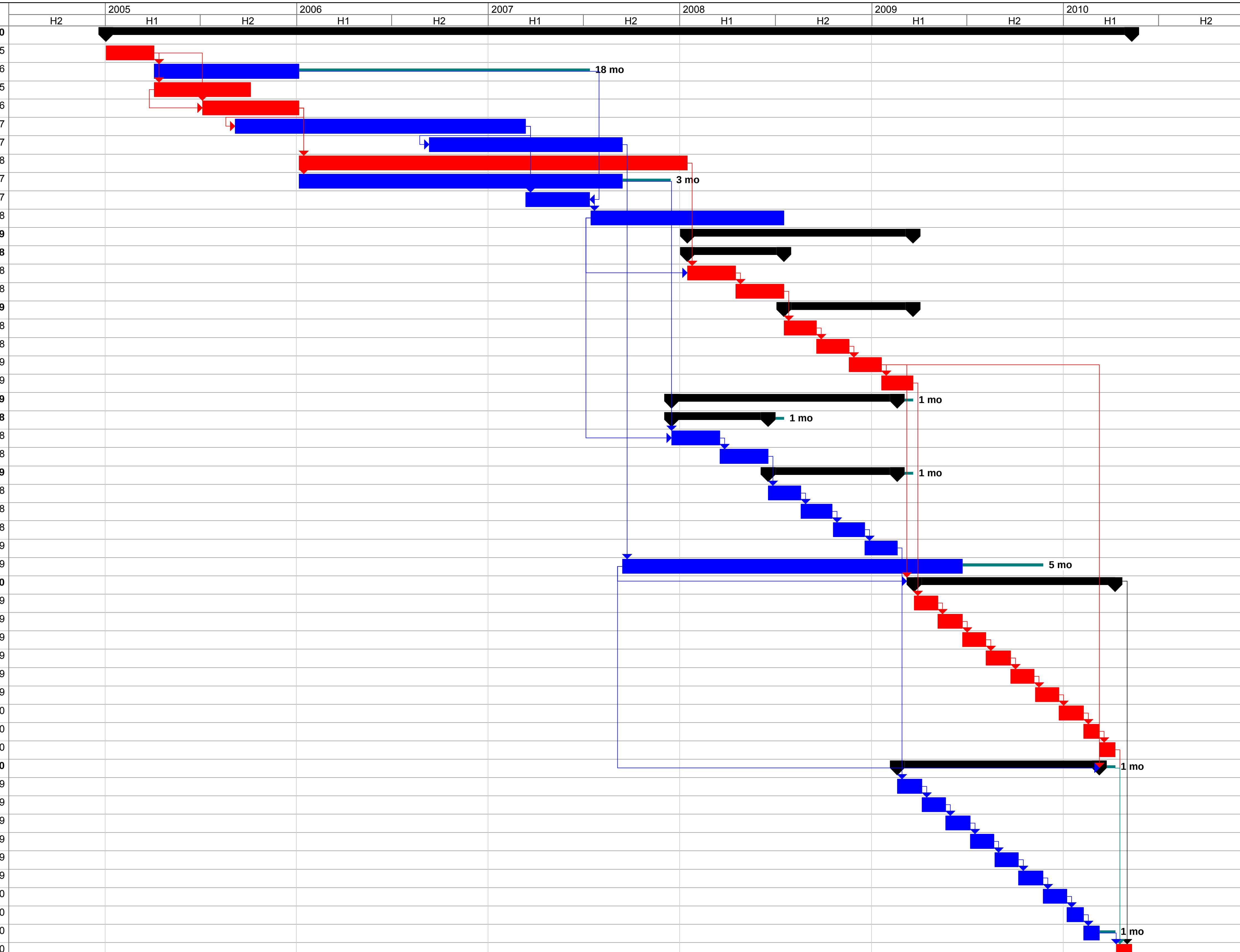
7.4 Construction Schedule:

The construction durations for the various elements of work are based on similar durations achieved on numerous cable-stayed bridges constructed or under construction in the United States. These durations have been confirmed by our constructability expert, Peter Sanderson.

The latest projected completion date for any of the alternates is mid-2011, which is in line with the completion date of the re-bid SAS.

Alternate 3, Scenario I (No Design Changes to T1 and E2 Foundations Implemented)

ID	Task Name	Duration	Finish
1	**Total Project**	**63.5 mo**	**5/11/10**
2	Decision Making, Design Consultant Contract	3 mo	4/4/05
3	EIS Revisions & Permits	9 mo	1/5/06
4	Additional Geotechnical Investigations	6 mo	10/5/05
5	Preliminary Design/Foundation Verification	6 mo	1/5/06
6	Tower & Superstructure Bid Package	18 mo	3/13/07
7	Superstructure Final Details	12 mo	9/13/07
8	T1 Tower Foundation Work	24 mo	1/15/08
9	E2 Tower Foundation Work	20 mo	9/13/07
10	Tower and Superstructure Bid Period	4 mo	7/13/07
11	Tower Shop Drawings & Submittal Prep.	12 mo	7/17/08
12	**West Tower Construction**	**14 mo**	**3/20/09**
13	**Lower Tower Construction**	**6 mo**	**7/17/08**
14	Lower Lift 1 - 4	3 mo	4/16/08
15	Lower Lift 5 - 8	3 mo	7/17/08
16	**Upper Tower Construction**	**8 mo**	**3/20/09**
17	Upper Lift 1 - 4	2 mo	9/17/08
18	Uppler Lift 5 - 8	2 mo	11/18/08
19	Upper Lift 9 - 12	2 mo	1/19/09
20	Upper Lift 13 - 16	2 mo	3/20/09
21	**East Tower Construction**	**14 mo**	**2/18/09**
22	**Lower Tower Construction**	**6 mo**	**6/17/08**
23	Lower Lift 1 - 4	3 mo	3/17/08
24	Lower Lift 5 - 8	3 mo	6/17/08
25	**Upper Tower Construction**	**8 mo**	**2/18/09**
26	Upper Lift 1 - 4	2 mo	8/18/08
27	Upper Lift 5 - 8	2 mo	10/17/08
28	Upper Lift 9 - 12	2 mo	12/18/08
29	Upper Lift 13 - 16	2 mo	2/18/09
30	Superstructure Shop Drawings & Fabrication	21 mo	6/22/09
31	**Superstructure Construction at West Tower**	**12.5 mo**	**4/9/10**
32	Segment 1 (Backspan and Main Span)	1.5 mo	5/6/09
33	Segment 2 (Backspan and Main Span)	1.5 mo	6/22/09
34	Segment 3 (Backspan and Main Span)	1.5 mo	8/6/09
35	Segment 4 (Backspan and Main Span)	1.5 mo	9/22/09
36	Segment 5 (Backspan and Main Span)	1.5 mo	11/6/09
37	Segment 6 (Backspan and Main Span)	1.5 mo	12/23/09
38	Segment 7 (Backspan and Main Span)	1.5 mo	2/8/10
39	Segment 8 (Main Span)	1 mo	3/10/10
40	Segment 9 (Main Span)	1 mo	4/9/10
41	**Superstructure Construction at East Tower**	**12.5 mo**	**3/10/10**
42	Segment 1 (Backspan and Main Span)	1.5 mo	4/6/09
43	Segment 2 (Backspan and Main Span)	1.5 mo	5/21/09
44	Segment 3 (Backspan and Main Span)	1.5 mo	7/7/09
45	Segment 4 (Backspan and Main Span)	1.5 mo	8/21/09
46	Segment 5 (Backspan and Main Span)	1.5 mo	10/7/09
47	Segment 6 (Backspan and Main Span)	1.5 mo	11/23/09
48	Segment 7 (Backspan and Main Span)	1.5 mo	1/7/10
49	Segment 8 (Main Span)	1 mo	2/8/10
50	Segment 9 (Main Span)	1 mo	3/10/10
51	Main Span Closure	1 mo	5/11/10

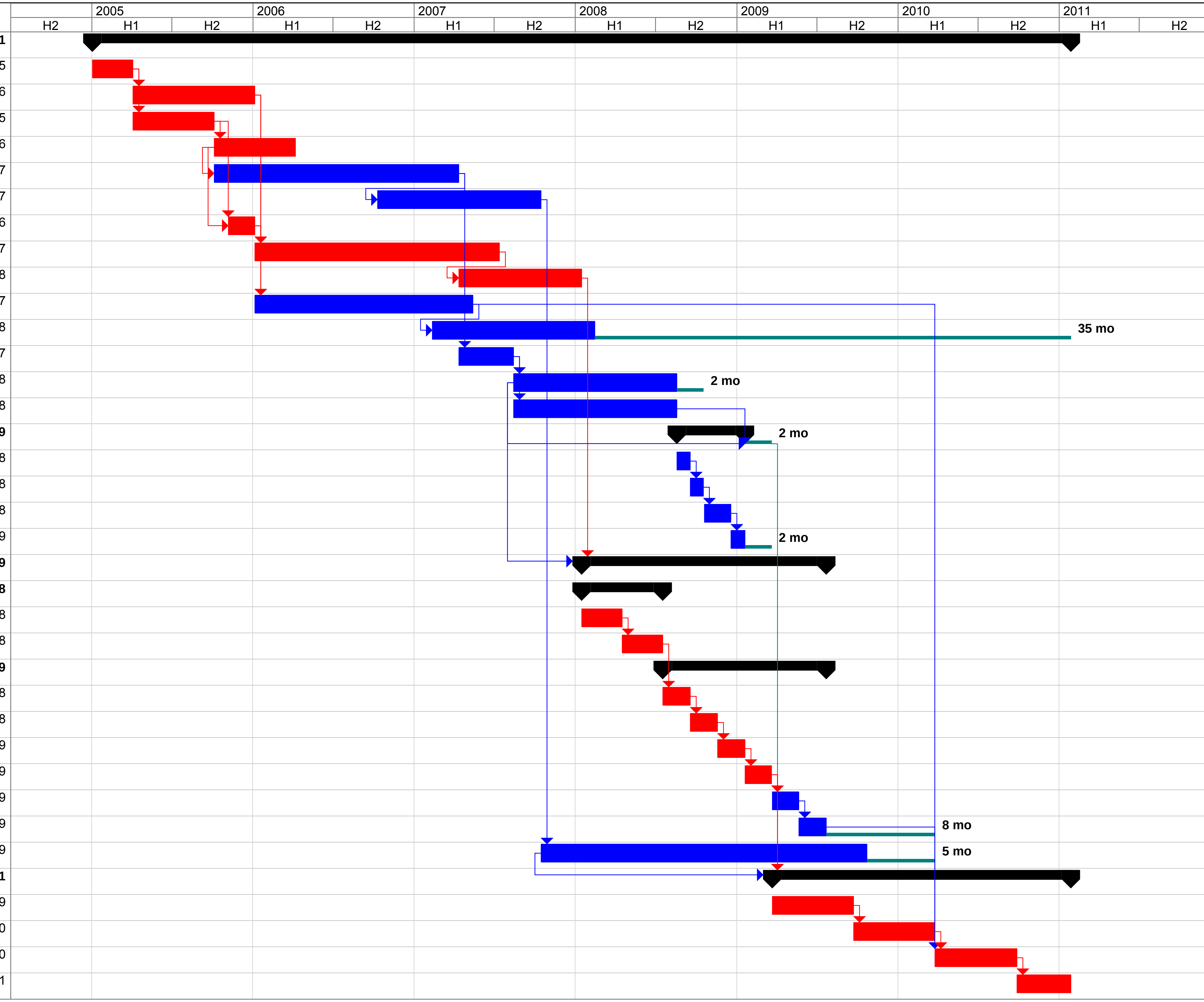


SFOBB - CS Alternate 3, Scenario I

Critical	Split	Slack	Project Summary	External Tasks			
Critical Split	Progress	Slippage	Rolled Up Critical	External Milestone			
Task	Milestone	Summary	Rolled Up Critical Split	Deadline			

Alternate I, Scenario II (Some Design Changes to T1 and E2 Required)

ID	Task Name	Duration	Finish
1	**Total Project**	**72 mo**	**1/27/11**
2	Decision Making, Design Consultant Contract	3 mo	4/4/05
3	EIS Revisions & Permits	9 mo	1/5/06
4	Preliminary Design	6 mo	10/5/05
5	Foundation Package	6 mo	4/7/06
6	Tower, W2, and Superstructure Bid Package	18 mo	4/12/07
7	Superstructure Final Details	12 mo	10/15/07
8	Foundation Negotiation with Kiewit	2 mo	1/5/06
9	T1 Tower Foundation Cofferdam, Piles, and Shafts	18 mo	7/13/07
10	T1 Tower Foundation Pile Cap Construction	9 mo	1/15/08
11	E2 Pier Foundation Cofferdam, Piles, and Shafts	16 mo	5/14/07
12	E2 Pier Foundation Pile Cap and Column Constuction	12 mo	2/14/08
13	Tower, W2, and Superstructure Bid Period	4 mo	8/14/07
14	Tower Shop Drawings & Submittal Prep.	12 mo	8/18/08
15	Pier W2 Construction	12 mo	8/18/08
16	**Concrete Backspan Construction**	**5 mo**	**1/19/09**
17	Construct Falsework	1 mo	9/17/08
18	Construct Bottom Slab	1 mo	10/17/08
19	Construct Webs & Floorbeams	2 mo	12/18/08
20	Construct Top Slab	1 mo	1/19/09
21	**Tower Construction**	**18 mo**	**7/22/09**
22	**Lower Tower Construction**	**6 mo**	**7/17/08**
23	Lower Lift 1 - 4	3 mo	4/16/08
24	Lower Lift 5 - 8	3 mo	7/17/08
25	**Upper Tower Construction**	**12 mo**	**7/22/09**
26	Upper Lift 1 - 4	2 mo	9/17/08
27	Uppler Lift 5 - 8	2 mo	11/18/08
28	Upper Lift 9 - 12	2 mo	1/19/09
29	Upper Lift 13 - 16	2 mo	3/20/09
30	Upper Lift 17 - 20	2 mo	5/21/09
31	Upper Lift 21 - 24	2 mo	7/22/09
32	Superstructure Shop Drawings & Fabrication	24 mo	10/22/09
33	**Superstructure Construction**	**22 mo**	**1/27/11**
34	Build Segment 1 - 4 (Main Span & Back Span)	6 mo	9/22/09
35	Build Segment 5 - 8 (Main Span & Back Span)	6 mo	3/25/10
36	Build Segment 9 - 12 (Main Span & Back Span)	6 mo	9/27/10
37	Build Segment 13 - 16 (Main Span Only)	4 mo	1/27/11

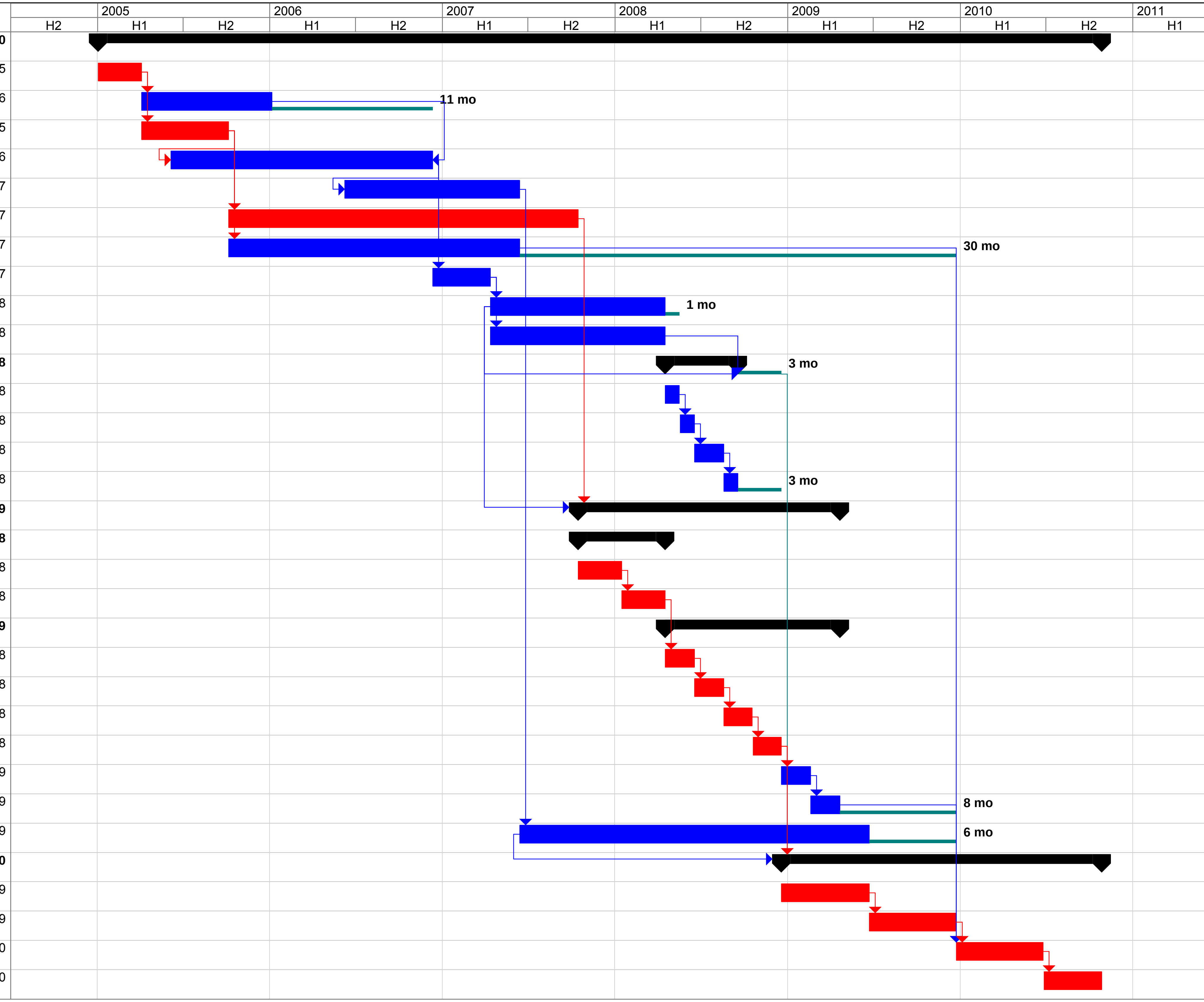


SFOBB - CS Alternate I, Scenario II

Critical	Split	Slack	Project Summary	External Tasks
Critical Split	Progress	Slippage	Rolled Up Critical	External Milestone
Task	Milestone	Summary	Rolled Up Critical Split	Deadline

Alternate I, Scenario I (No change to T1 and E2 Foundations Required)

ID	Task Name	Duration	Finish
1	**Total Project**	**69 mo**	**10/27/10**
2	Decision Making, Design Consultant Contract	3 mo	4/4/05
3	EIS Revisions & Permits	9 mo	1/5/06
4	Preliminary Design/Foundation Verification	6 mo	10/5/05
5	Tower, W2, & Superstructure Bid Package	18 mo	12/11/06
6	Superstructure Final Details	12 mo	6/13/07
7	T1 Tower Foundation Work	24 mo	10/15/07
8	E2 Pier Foundation Work	20 mo	6/13/07
9	Tower, W2, and Superstructure Bid Period	4 mo	4/12/07
10	Tower Shop Drawings & Submittal Prep.	12 mo	4/16/08
11	Pier W2 Construction	12 mo	4/16/08
12	**Concrete Backspan Construction**	**5 mo**	**9/17/08**
13	Construct Falsework	1 mo	5/16/08
14	Construct Bottom Slab	1 mo	6/17/08
15	Construct Webs & Floorbeams	2 mo	8/18/08
16	Construct Top Slab	1 mo	9/17/08
17	**Tower Construction**	**18 mo**	**4/21/09**
18	**Lower Tower Construction**	**6 mo**	**4/16/08**
19	Lower Lift 1 - 4	3 mo	1/15/08
20	Lower Lift 5 - 8	3 mo	4/16/08
21	**Upper Tower Construction**	**12 mo**	**4/21/09**
22	Upper Lift 1 - 4	2 mo	6/17/08
23	Uppler Lift 5 - 8	2 mo	8/18/08
24	Upper Lift 9 - 12	2 mo	10/17/08
25	Upper Lift 13 - 16	2 mo	12/18/08
26	Upper Lift 17 - 20	2 mo	2/18/09
27	Upper Lift 21 - 24	2 mo	4/21/09
28	Superstructure Shop Drawings & Fabrication	24 mo	6/22/09
29	**Superstructure Construction**	**22 mo**	**10/27/10**
30	Build Segment 1 - 4 (Main Span & Back Span)	6 mo	6/22/09
31	Build Segment 5 - 8 (Main Span & Back Span)	6 mo	12/23/09
32	Build Segment 9 - 12 (Main Span & Back Span)	6 mo	6/25/10
33	Build Segment 13 - 16 (Main Span Only)	4 mo	10/27/10

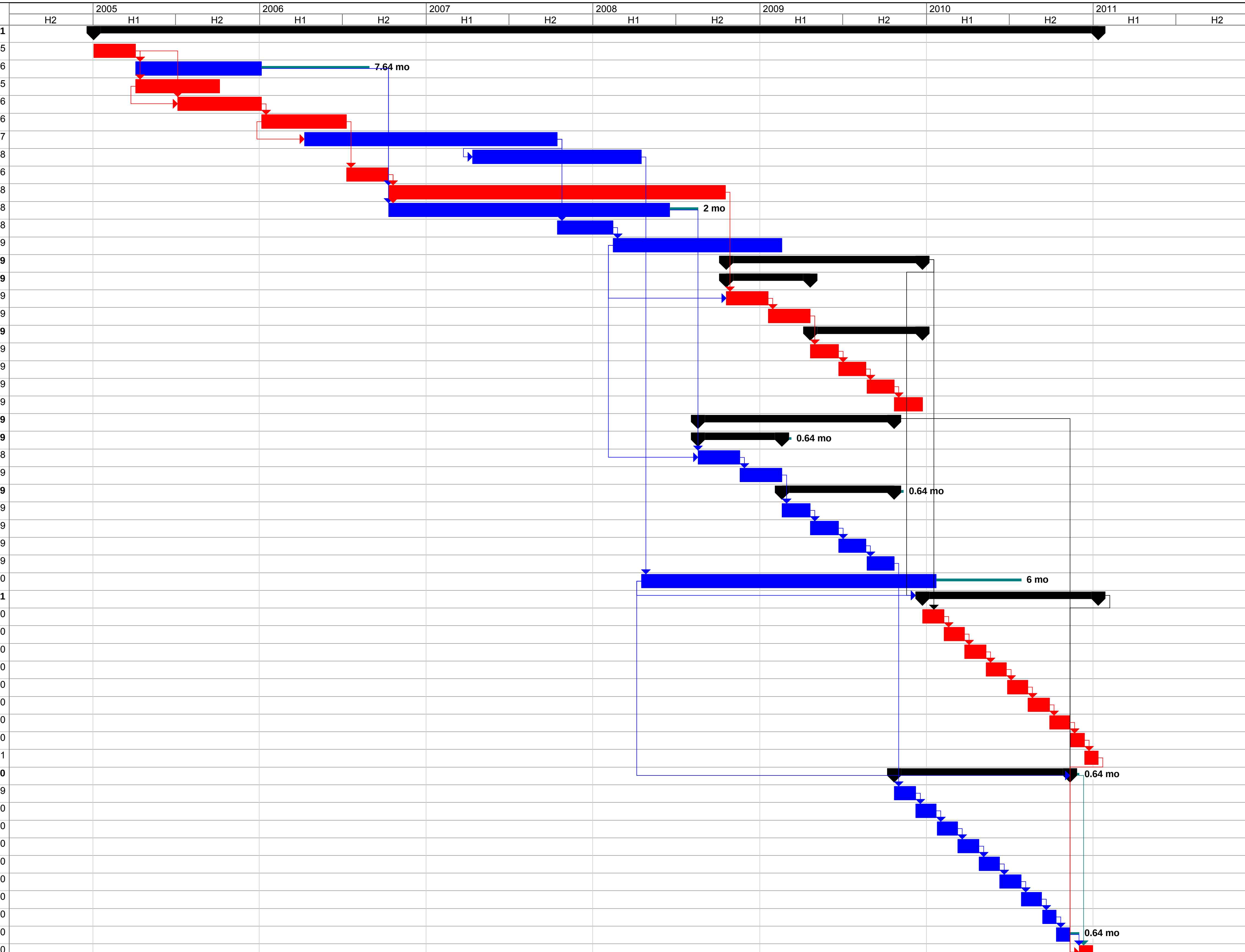


SFOBB - CS Alternate I, Scenario I

Critical		Split		Slack		Project Summary		External Tasks	
Critical Split		Progress		Slippage		Rolled Up Critical		External Milestone	
Task		Milestone		Summary		Rolled Up Critical Split		Deadline	

Alternate 3, Scenario II (Design Change to T1 and E2 Implemented To Reduce Foundation Sizes)

ID	Task Name	Duration	Finish
1	**Total Project**	**71.5 mo**	**1/12/11**
2	Decision Making, Design Consultant Contract	3 mo	4/4/05
3	EIS Revisions & Permits	9 mo	1/5/06
4	Additional Geotechnical Investigations	6 mo	10/5/05
5	Preliminary Design	6 mo	1/5/06
6	Foundation Package	6 mo	7/10/06
7	Tower & Superstructure Bid Package	18 mo	10/15/07
8	Superstructure Final Details	12 mo	4/16/08
9	Foundation Bid Period	3 mo	10/10/06
10	T1 Tower Foundation Work	24 mo	10/17/08
11	E2 Tower Foundation Work	20 mo	6/17/08
12	Tower and Superstructure Bid Period	4 mo	2/14/08
13	Tower Shop Drawings & Submittal Prep.	12 mo	2/18/09
14	**West Tower Construction**	**14 mo**	**12/23/09**
15	**Lower Tower Construction**	**6 mo**	**4/21/09**
16	Lower Lift 1 - 4	3 mo	1/19/09
17	Lower Lift 5 - 8	3 mo	4/21/09
18	**Upper Tower Construction**	**8 mo**	**12/23/09**
19	Upper Lift 1 - 4	2 mo	6/22/09
20	Uppler Lift 5 - 8	2 mo	8/21/09
21	Upper Lift 9 - 12	2 mo	10/22/09
22	Upper Lift 13 - 16	2 mo	12/23/09
23	**East Tower Construction**	**14 mo**	**10/22/09**
24	**Lower Tower Construction**	**6 mo**	**2/18/09**
25	Lower Lift 1 - 4	3 mo	11/18/08
26	Lower Lift 5 - 8	3 mo	2/18/09
27	**Upper Tower Construction**	**8 mo**	**10/22/09**
28	Upper Lift 1 - 4	2 mo	4/21/09
29	Upper Lift 5 - 8	2 mo	6/22/09
30	Upper Lift 9 - 12	2 mo	8/21/09
31	Upper Lift 13 - 16	2 mo	10/22/09
32	Superstructure Shop Drawings & Fabrication	21 mo	1/22/10
33	**Superstructure Construction at West Tower**	**12.5 mo**	**1/12/11**
34	Segment 1 (Backspan and Main Span)	1.5 mo	2/8/10
35	Segment 2 (Backspan and Main Span)	1.5 mo	3/25/10
36	Segment 3 (Backspan and Main Span)	1.5 mo	5/11/10
37	Segment 4 (Backspan and Main Span)	1.5 mo	6/25/10
38	Segment 5 (Backspan and Main Span)	1.5 mo	8/11/10
39	Segment 6 (Backspan and Main Span)	1.5 mo	9/27/10
40	Segment 7 (Backspan and Main Span)	1.5 mo	11/11/10
41	Segment 8 (Main Span)	1 mo	12/13/10
42	Segment 9 (Main Span)	1 mo	1/12/11
43	**Superstructure Construction at East Tower**	**12.5 mo**	**11/11/10**
44	Segment 1 (Backspan and Main Span)	1.5 mo	12/8/09
45	Segment 2 (Backspan and Main Span)	1.5 mo	1/22/10
46	Segment 3 (Backspan and Main Span)	1.5 mo	3/10/10
47	Segment 4 (Backspan and Main Span)	1.5 mo	4/26/10
48	Segment 5 (Backspan and Main Span)	1.5 mo	6/10/10
49	Segment 6 (Backspan and Main Span)	1.5 mo	7/27/10
50	Segment 7 (Backspan and Main Span)	1.5 mo	9/10/10
51	Segment 8 (Main Span)	1 mo	10/12/10
52	Segment 9 (Main Span)	1 mo	11/11/10
53	Main Span Closure	1 mo	12/31/10

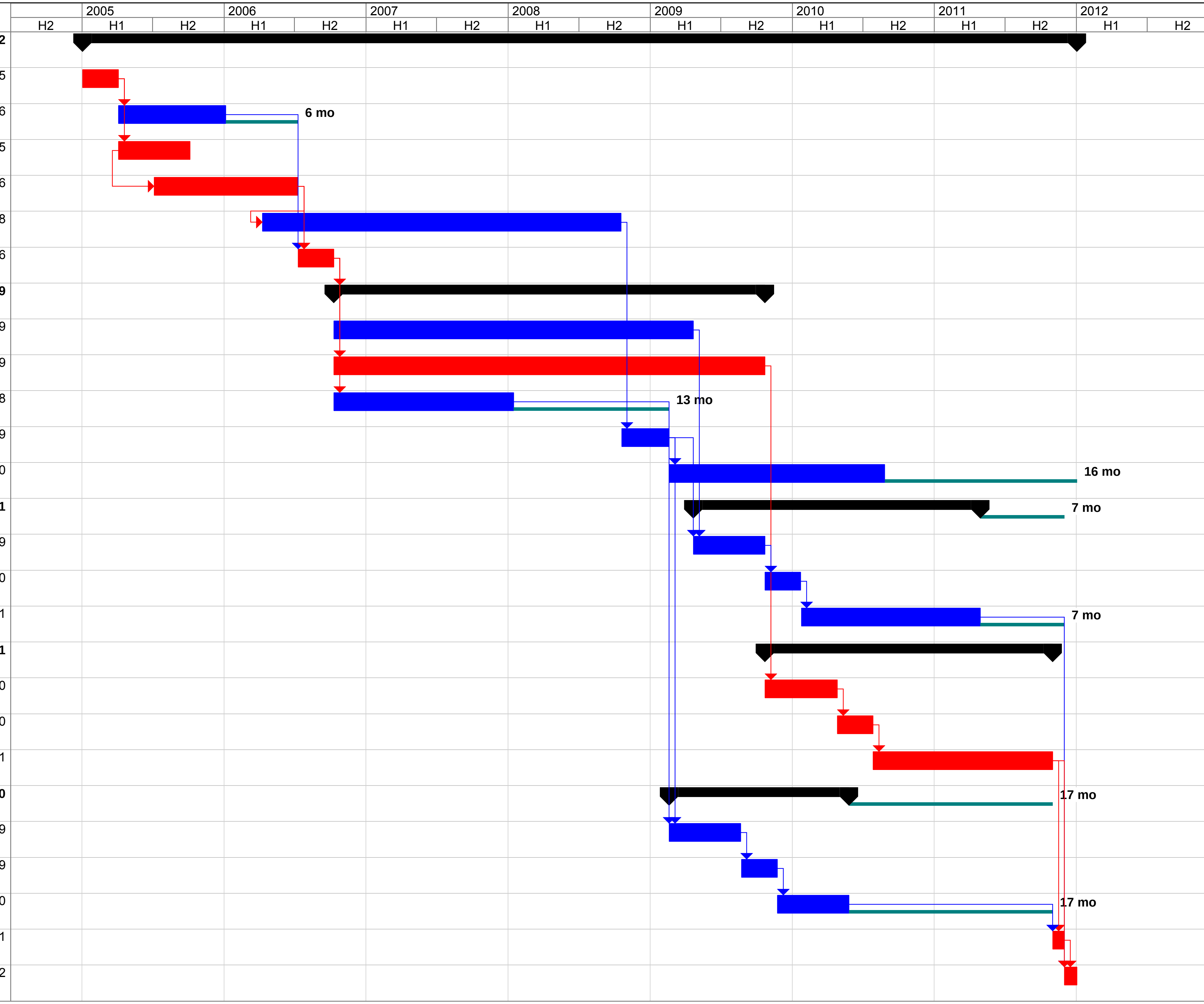


SFOBB - CS Alternate 3, Scenario II

Critical	Split	Slack	Project Summary	External Tasks
Critical Split	Progress	Slippage	Rolled Up Critical	External Milestone
Task	Milestone	Summary	Rolled Up Critical Split	Deadline

Skyway Scenario

ID	Task Name	Duration	Finish
1	**Total Project**	**83 mo**	**1/2/12**
2	Decision Making, Design Consultant Contract	3 mo	4/4/05
3	EIS Revisions & Permits	9 mo	1/5/06
4	Additional Geotechnical Investigation	6 mo	10/5/05
5	Foundation Design	12 mo	7/10/06
6	Pier and Superstructure Design	30 mo	10/17/08
7	Foundation Bid Period	3 mo	10/10/06
8	**Foundation Construction Activities**	**36 mo**	**10/22/09**
9	T1 Foundation Construction	30 mo	4/21/09
10	E2A Foundation Construction	36 mo	10/22/09
11	E2 Foundation Construction	15 mo	1/15/08
12	Pier and Superstructure Bid Period	4 mo	2/18/09
13	Shop Drawings and Erection Manual	18 mo	8/26/10
14	**T1 Construction Activities**	**24 mo**	**4/29/11**
15	Pier T1 Construction	6 mo	10/22/09
16	Pier T1 Table Construction	3 mo	1/22/10
17	T1 Superstructure Construction	15 mo	4/29/11
18	**E2A Construction Activities**	**24 mo**	**11/1/11**
19	Pier E2A Construction	6 mo	4/26/10
20	Pier E2A Table Construction	3 mo	7/27/10
21	E2A Superstructure Construction	15 mo	11/1/11
22	**E2 Construction Activities**	**15 mo**	**5/26/10**
23	Pier E2 Construction	6 mo	8/21/09
24	Pier E2 Table Construction	3 mo	11/23/09
25	E2 Superstructure Construction	6 mo	5/26/10
26	Closure E2A/E2	1 mo	12/1/11
27	Closure E2A/T1	1 mo	1/2/12



SFOBB - Skyway Scenario

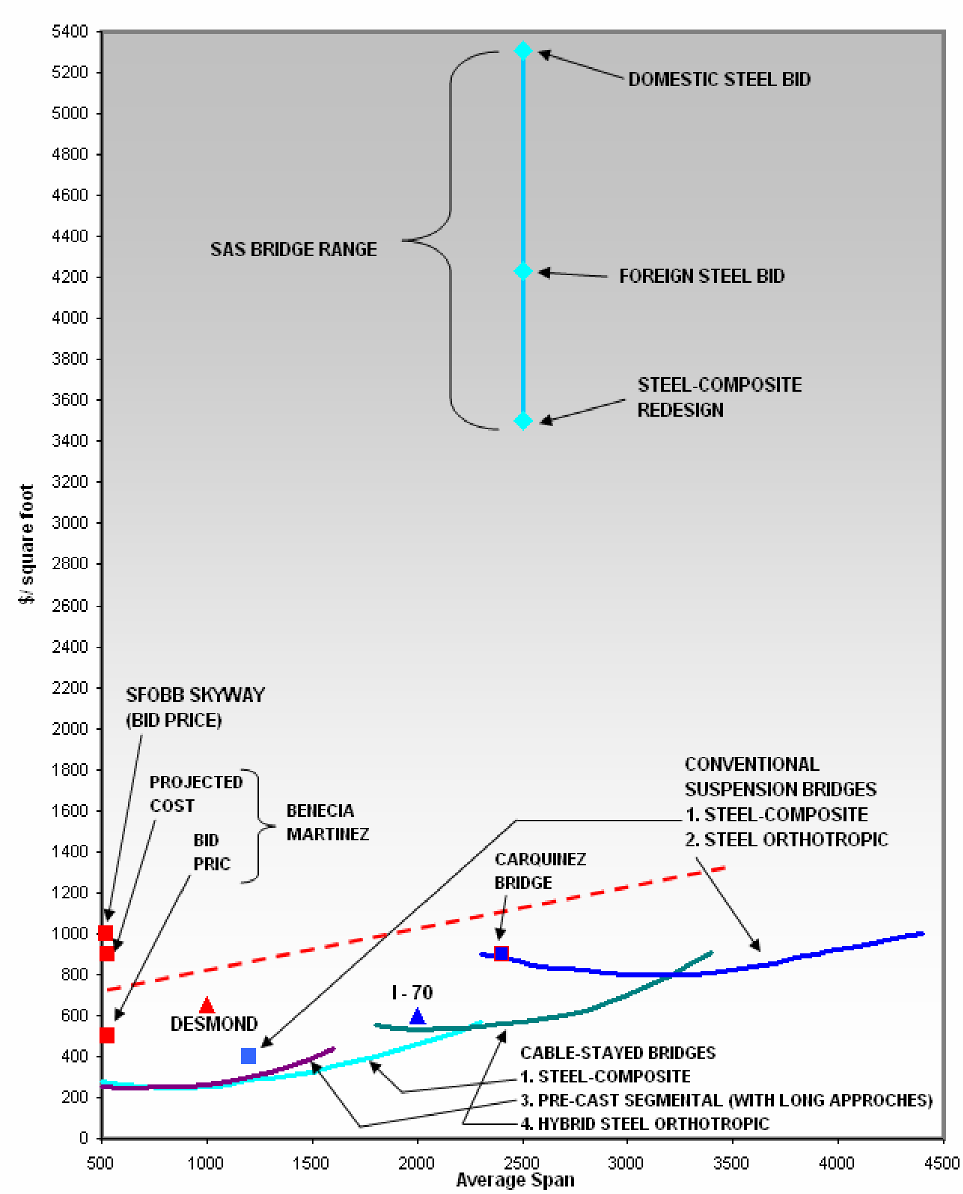
Critical	Split	Slack	Project Summary	External Tasks	
Critical Split	Progress	Slippage	Rolled Up Critical	External Milestone	
Task	Milestone	Summary	Rolled Up Critical Split	Deadline	

Estimated Cost Savings

8. ESTIMATED COST SAVINGS

8.1 Background

Cost savings presented in the IRT preliminary findings were based on historical data for cable-stayed bridges, adjusted for the environment of the SFOBB. The chart below was used by the IRT to estimate that the cable-stayed design could save in excess of $500,000,000. As a result of concern by Caltrans staff that this might not accurately represent the design for a cable-stayed bridge at this specific location, Peter Sanderson, a construction expert with significant experience bidding and building long span structures including bridges in California was added to the IRT to independently estimate the cable-stayed bridges.



8.2 Cost Estimate Summary

Peter Sanderson prepared construction cost estimates for both Alternate 1 and Alternate 3 using contractor estimating methodology. The estimates are based on updated quantities from the Phase 2 seismic analysis studies. The following table provides a summary of costs of the various construction contracts to provide direct comparison to the anticipated cost (based on bid results) of the SAS bridge. Design cost as well as an estimated cost for the delay to the Kiewit E2/T1 contract is included. These estimates were escalated to midpoint of construction.

The estimates developed by Mr. Sanderson confirmed the potential construction savings identified in the Preliminary Recommendations of the IRT.

Appendix A provides the details of the estimates.

Cost Estimated Savings Comparison – SAS Bid vs. Cable-Stayed Options

	Option						
	1	2a		2b		3	
Description	Award Current Contract	Readvertise Contract in January 2005		Readvertise Contract in September 2005		Redesign & Bid Cable Stayed Bridge Design	
	To	From	To	From	To	From	To
Substructure - W2							
Original Contract Amount	24,083,285	24,083,285	24,083,285	24,083,285	24,083,285	24,083,285	24,083,285
Resolved (Approved) CCOs	320,119	320,119	320,119	320,119	320,119	320,119	320,119
Unresolved CCOs	82,197	82,197	82,197	82,197	82,197	82,197	82,197
Issued/Unresolved NOPCs	1,485,529	1,485,529	1,485,529	1,485,529	1,485,529	1,485,529	1,485,529
State Furnished Material	125,200	125,200	125,200	125,200	125,200	125,200	125,200
W2 Modifications						Note ***	Note ***
Substructure - E2/T1							
Original Contract Amount	177,450,000	177,450,000	177,450,000	177,450,000	177,450,000	125,000,000	177,450,000
Tower & East Pier Construction Cost						Note ***	Note ***
Resolved (Approved) CCOs	-	-	-		-		
Unresolved CCOs	1,389,000	1,389,000	1,389,000	1,389,000	1,389,000	1,389,000	1,389,000
Issued/Unresolved NOPCs	-	-	-	-	-	-	-
Pending Changes	8,549,000	8,549,000	8,549,000	8,549,000	8,549,000	8,549,000	8,549,000
State Furnished Material	328,000	328,000	328,000	328,000	328,000	328,000	328,000
Bridge Superstructure							
Original Contract Amount	1,398,776,550	1,398,776,550	1,398,776,550	1,398,776,550	1,398,776,550		
Cable Stayed Bridge Construction Cost (see Appendix A)						364,709,122	450,579,458
Miscellaneous Items (see Appendix A)						65,000,000	65,000,000
State Furnished Material	3,000,000	3,000,000	3,000,000	3,000,000	3,000,000	3,000,000	3,000,000
Cable Stayed Bridge TRO (see appendix)						67,649,056	85,265,953
Cable Stayed Bridge Design Cost						25,000,000	25,000,000
Subtotal - Bridge Construction Cost	1,615,588,880	1,615,588,880	1,615,588,880	1,615,588,880	1,615,588,880	686,720,508	842,657,741
Current E2 / T1 Contract Sunk Costs:							
Cost to-date						-	-
Additional Cost to Contract Cancellation						-	-
Structural Steel Ordered						-	-
Other Material Ordered						-	-
Demobilization						-	-
Compensation for Contract Cancellation						-	-
COS							
Delay to E2/T1 (8 months)						60,000,000	60,000,000
Current E2 / T1 Contract Sunk Costs:						-	-
Impact on Adjacent Contract							
South South Detour Termination		20,000,000	25,000,000	20,000,000	25,000,000		
Rebid South South Detour, escalation (2 yr@5%/yr) & Uncertainties (20%)		20,000,000	20,000,000	20,000,000	20,000,000		
Skyway Modifications							
Escalation for impacted contracts (YBI Transition, Oakland Touchdown, Demolition) 5% per year		39,000,000	66,000,000	39,000,000	66,000,000	-	-
YBI Impact						-	-
Impact on Adjacent Contract		79,000,000	111,000,000	79,000,000	111,000,000	-	-
Bid Competition		(150,000,000)	(75,000,000)	(150,000,000)	(75,000,000)		
Contract Improvement Savings		(40,000,000)	(20,000,000)	(40,000,000)	(20,000,000)		
Subtotal	1,615,588,880	1,504,588,880	1,631,588,880	1,504,588,880	1,631,588,880	686,720,508	842,657,741
Escalation		76,000,000	76,000,000	126,000,000	126,000,000	-	-
Subtotal	1,615,588,880	1,580,588,880	1,707,588,880	1,630,588,880	1,757,588,880	686,720,508	842,657,741
Contingency						100,000,000	100,000,000
Capital Cost Total (Excluding contingency & Potential Future Costs)	1,615,588,880	1,580,588,880	1,707,588,880	1,630,588,880	1,757,588,880	786,720,508	942,657,741
Increase from Option 1: W/O Future Costs		(35,000,000)	92,000,000	15,000,000	142,000,000	(828,868,372)	(672,931,139)
Potential Future Costs	350,000,000	350,000,000	350,000,000	350,000,000	350,000,000	100,000,000	100,000,000
Increase from Option 1: with Future Costs		(35,000,000)	92,000,000	15,000,000	142,000,000	1,078,868,372	887,931,139
*** Included in bridge cost							

SAS Risk Review

9. SELF ANCHORED SUSPENSION BRIDGE RISK REVIEW

One of the elements of the SAS bridge that the IRT was asked to review concerned the risk characteristics associated with its construction. In doing so, Mr. Peter Sanderson (see enclosed resume at Appendix A) was asked to consider this question in light of his 35 years of experience in building and bidding large projects. The following summarizes Mr. Sanderson's analysis of the risks associated with constructing the SAS Bridge as designed and bid on May 26, 2004.

9.1 General Comments

During the long period that the self-anchored suspension (SAS) bridge was out to bid, a number of outreach meetings were held between Caltrans and the construction industry. In addition, some 783 Requests for Information (RFI) were submitted and then answered in some fashion, and in many cases not to the satisfaction of those who posed the questions. Despite the many meetings and the large number of RFIs, most people involved in bidding the SAS still have a sense that there never was closure on many of the subjects raised, and many expected trouble on a number of matters if the project had gone on to the construction phase.

Mr. Sanderson reviewed all the RFIs and their responses, looked at the majority of the drawings, and read relevant specification sections. This narrative contains his observations about fabrication and construction based on these reviews and this reading. Much of what follows should not be taken as statements of fact, but rather observations based on limited direct knowledge and some hearsay. Obviously the observations are grounded on his experience in building other than SAS bridges. Mr. Sanderson is not among the tiny number of people who have been involved in construction of an SAS bridge. However, his relevant experience covers several long-span steel bridges, one suspension bridge, a number of moveable bridges, and a large number of cable-stayed bridges.

Difficulties start with the enormous size of pre-assembled sections. Both the deck and the tower need to be welded into pieces so big that fabrication is limited to shipyards, and further limited to a very small number of shipyards when you look at potential suppliers worldwide. While a bid was received in May for the project using domestic fabrication, Mr. Sanderson cannot be sure that this number was based on a quote received from domestic fabricators. Most probably this bid was submitted only to demonstrate that domestic steel was in excess of 25% more expensive, which then allows consideration of imported steel.

One experienced fabricator did look into building a fabrication facility in Alameda County, but abandoned that pursuit when calculations of the cost were completed. Costs were estimated at over $100 million. A joint venture of fabricators from the Pacific Northwest did pursue the SAS for a long time, but eventually declined to bid. Please note that for the recently completed Carquinez Bridge (also an orthotropic deck suspension bridge), the bid from this Pacific Northwest group was double that of the fabricator who did eventually build the deck sections.

The contractor thus will be restricted to fabricators such as Samsung, IHI, Kawada, and Mitsubishi as sources of fabricated steel. This is because of the large assemblies to be welded up and the specification of submerged arc welding. Submerged Arc Welding must be done in the flat, down position, so pieces must be constantly rotated. Rotation requires huge overhead crane capacity, which only these fabricators have.

Samsung, Kawada, and the like have a wealth of experience with large steel fabrications, but are unused to working with the specific requirements and specifications written for this project. Perhaps IHI should be listed as an exception because of their experience as fabricator for the Carquinez Bridge. Risk arises because an impasse is often reached between the people who have done an awful lot of this type of work and those who have written the specifications, who are often less experienced. Delay and claims are usually the end result.

Had the SAS been awarded earlier this year, fabrication was scheduled for 2006, 2007, and 2008. During all that time it could have been said that might makes right for the fabricator, because he can refuse to ship. The contractor can, of course, refuse to pay, but will probably have posted letters of credit to cover such eventualities, and is vulnerable. Payment disputes thus open another major delay channel. There are rumors that delays are occurring on a suspension bridge in Washington State because the fabricator is, in a way, holding steel "hostage" while a payment dispute is sorted out.

During the course of fabrication of the Carquinez Bridge, many disputes between the designer and the fabricator were resolved with the help of mock-ups or models. To be more specific, the fabricator often stated that certain welds could not be made as designed and built mock-ups to help prove it. The SAS requires first plywood or Styrofoam mock-ups at half size, then steel mock-ups at full size. This is alarming because it indicates that the designer is nervous about whether or not certain welds can be made, and is hoping to have his problems unearthed early on during the fabrication of the mock-ups. Please note that the wood mock-up needs approval before starting on drawings for the steel mock-up, and that the steel mock-up needs approval before fabrication drawings can be started.

Many delays are likely throughout this process, especially when you notice that one of the would-be bidders, in RFI # 706, stated that at least 60 questions were at that late date unanswered. The fact that this project went through a very long bid period the first time around does not mean that all the problems have been unearthed. Indeed, that so many were asked is just indicative of the enormous number to come. There is no known ratio of problems unearthed pre-bid to problems discovered during the construction period, but there is certainly a strong correlation between the two. The Carquinez Bridge saw RFIs issued for two years before start up of fabrication, but had many fewer questions (than the SAS) asked during the bid period.

Additional difficulties can be foreseen with the various facets of this project as will be described in detail below. All of these could be resolved in a timely manner, but experience with Caltrans shows otherwise. Delay and claims will be the result.

9.2 Fabrication

If the fabrication of the different bridge components is difficult or nearly impossible to achieve to the specifications of the contract, then delays and additional costs are incurred. The following observations are made with respect to the fabrication activities associated with the SAS:

1. Tower vertical stiffeners cannot be properly welded into position after adjacent stiffeners are in place.

2. When diagonal stiffener plates at the corners are welded into position, this weld cannot be radiographed because the film plate cannot be recovered.
3. The saddle castings are so big, that there is only one caster in the world that can do the job. This caster has called the machining requirements “very onerous.”
4. Requirements in the contract, such as the one calling for each working drawing to reference the Contract Plan from which “fabricable dimensions are derived,” will add a great deal of time to the production process. Questions from the designer will slow down the detailer, and probably slow the overall progress.
5. Tolerances of the fabricated panel structure are also required on fabrication drawings. This will also result in disputes because prediction of lengths after welding is very difficult.
6. Submission of details of fabrication jigs and measurement templates has not been done in the past.
7. Submission of tack weld details is unprecedented.
8. Submission of details of temporary work platforms is unprecedented.
9. Calculations indicating stresses due to attachments and to transportation are unprecedented.
10. The Weight Control Procedure requirement raises a red flag—rolling tolerances may well lead to big problems. Plates that are within tolerance before fabrication begins may result in the final product being out of tolerance. What happens next is unknown.
11. There is a requirement for all fabrications to be true at average bridge temperature. Does this mean that every cut or weld will be moved, depending on the temperature at the time?
12. The orthotropic rib design has never been used before, and will probably be extremely difficult to keep in dimensional tolerance.
13. Detail drawings cannot be started until cambers are done by the erector and approved by the Engineer. Many battles await on this subject, and delays will result. Preparation of details is now solidly on the critical path, and there are many, many obstacles to starting the drawings—never mind completing them.
14. The 2mm rounded corner requirement will lead to disputes, and this doesn’t help with paint adherence.
15. Welding pre-heat and grinding requirements are sure to be controversial.

9.3 Erection

The erection of the various bridge components is another area where there are considerable problems with the SAS bridge as designed and described in the contract for this project. Below are examples of the problems that have been identified:

1. The tower top section weighs over 500 tons and needs to be placed 160 meters in the air. There is no crane that can do this. The contractor will have to design a very sophisticated tilting mast assembly, which is at least 160 meters high and sits on its own foundation. This is an unprecedented lift, and the refinery construction business has a history of problems with this type of vessel tilting operation.
2. Once a mast pair is tilted away from a vertical orientation, capacity is lost rapidly, and in this case the masts must be tilted to pick up the top of the tower and avoid the base.
3. The 70mm splice plates will be very difficult to handle because of their sheer size.

4. The bottom part of the tower can be handled by a crane based in the Gulf of Mexico. However, this must be booked well in advance, and any delays on this site would see the crane depart as scheduled, whether or not the work is complete.
5. Tower aerodynamic stability has been questioned, and there is no reassurance that problems can be easily resolved. Given the location, stabilizing cables may be impractical.
6. The deck will require approximately 50,000 tons of falsework. All sorts of problems with seismic and wind loads may arise. Design and erection of the falsework, taken on its own, would be one of the biggest structural steel erection jobs undertaken in recent years in this country.
7. Given the location of the bridge and the stringent settlement and camber requirements, all major legs of this falsework will need to be supported at the bedrock layer, up to 90 meters below water level. Each leg will need a number of 2400 to 3000 diameter steel shell piles. Recent California experience with such size piles includes a lot of delay.
8. The erection requirements listed on Sheet 533 are extremely onerous and are probably unrealistic. There is a good chance they cannot be achieved.
9. The sponsor of the joint venture that was the only bidder in May has an Engineering Department consisting of only two engineers and a number of designers.

9.4 Concrete

The concrete elements of the bridge present some additional difficulties. Two of them are:

1. The W2 Cap Beam is an extremely large pour. There are vertical, longitudinal, and transverse tendons. Additionally, there are many layers of large diameter rebar, tie down ducts, and also a need for cooling water ducts because this is mass concrete. The probability of it all fitting is low.

2. The probability of the tower base fitting on the bolts installed by the foundation contractor is also low.

9.5 Cables

The cables required for the SAS are unique and carry their own set of unique challenges. Some are noted below:

1. Suspenders are required to be jacked into place. This will be extremely difficult.
2. PWS strands cannot be accurately made because of the different length wires needed to ensure equal tension. This will be a major dispute item with the contractor.
3. There is little experience with the S-wire wrapping process. There is also only one supplier in the world. New processes introduce risk and will likely result in claims and/or delays.

9.6 Painting

The painting requirements for this project offer additional risks to the contractor and Caltrans. Some of these are:

1. The specified water-based inorganic zinc primer will cause problems with field painting.
2. Given the atmosphere in San Francisco, delays could arise from the field coating process.

9.7 Allowance Recommendation

Considering all the risks listed above, the owner should set aside a large amount of money and time to cover changes. The preparation of shop drawings is firmly on the critical path, and it is this activity, which will be delayed that will ultimately delay the overall project as the various problems with the design are unearthed. The following is a summary of cost and time impacts that should be anticipated in moving ahead with the SAS bridge:

1. An additional six months' time and twenty percent of the fabrication cost should be allowed. This twenty percent of fabrication cost is approximately $88,000,000.
2. An additional ten percent of field construction costs should also be allowed, which amounts to approximately $90,000,000.
3. One year's worth of Time Related Overhead (TRO) will be approximately $33,000,000.

The total of these three items is $ 211,000,000.

All of the above assumes that just a series of minor problems comes up, and that those problems are resolved expeditiously. If far more serious problems occur, then these dollar values will be inadequate and additional time will be required to complete the project. ***This could result in several years of delay in opening the bridge.*** In this case, the contractor may be able to claim and win delay charges much greater than the Contractual Time Related Overhead. This is likely because the bidders repeatedly informed the owner during the tender period that the TRO rate allowed was inadequate.

9.8 Recommendation

With the forgoing in mind, and looking at the Benicia-Martinez experience and remembering the years of delay at Carquinez, it is recommended that a contingency in the order of **$350,000,000** be added to this project.

Environmental Review

10. ENVIRONMENTAL REVIEW

10.1 Background

The IRT evaluated the implications of switching from a SAS to a cable-stayed design from an environmental impact perspective. Such an evaluation is a critical component of any decision to modify the design given the sensitive nature of the project setting (i.e., the San Francisco Bay ecosystem). David J. Powers & Associates is an environmental expert on bridge and highway projects in the Bay Area.

The environmental process surrounding the East Span of the SFOBB has been thorough and extensive in its outreach to the public and numerous stakeholders. The IRT has been impressed with the level of effort demonstrated by all in moving the project through this process to complete the Environmental Impact Statement (EIS), as well as securing the Record of Decision (ROD) from the Federal Highway Administration (FHWA).

This process has taken years to complete. While a cable-stayed bridge was one of the many alternatives originally considered, the SAS concept and design were ultimately advanced as the locally preferred alternatives for the project. At the time these decisions were being made, the understanding of the substantial cost differential between an SAS design and a cable-stayed alternative was not available.

The IRT understands that the EIS process does not consider cost as an element in making a decision for the locally preferred alternative. However, when two essentially equal alternatives progress through the process with similar environmental impacts, public policy makers involved in selecting between alternative can and should consider the fiscal implications.

A number of possible environmental impacts may result from advancing a cable-stayed design. In order to gain an understanding of the consequences of a redesigned bridge for the main span, the information provided through the technical analysis found in Section V was analyzed in detail and conclusions drawn. Appendix C contains a letter from John Hesler of David J. Powers & Associates articulating his expert views on the impacts a cable-stayed bridge option would have. It should be noted that Mr. Hesler believes the impacts to be relatively minor and easily addressed in a nine-month period, which would occur simultaneously with engineering analysis requisite for moving ahead with a redesigned bridge.

10.2 Conclusions

In the context of the entire project, previous EIS work, and existing permits, changing to a cable-stayed alternative should not pose a significant change to the environmental document. It must be remembered that the criterion for evaluating changes is not just the main span, but the entire project as a whole. The IRT evaluation concluded the following:

- Both the SAS and cable-stayed designs were fully evaluated as design options under the Preferred Alternative in the SFOBB's Final Environmental Impact Statement (FEIS) that was completed in 2001. The FEIS concluded that the overall environmental impacts of these two options were virtually identical.

- The visual impacts of the cable-stayed design would be similar to those of the SAS design.
- Long-term impacts to the bay for the SAS design would be almost identical to those of cable-stayed design Alternatives 1 and 3. Short-term impacts to the bay under the cable-stayed design would likely be less than that of the SAS design, since the need for temporary piers would be avoided.
- Changing to a cable-stayed design would ***not*** require lengthy additional environmental studies. Additional documentation under NEPA could be accomplished with a reevaluation.
- Changing to a cable-stayed design would not require major modifications to existing permits.
- All environmental tasks related to changing from a SAS to a cable-stayed design can be accomplished in a 9-month period.
- For Alternates 1 and 3, since the foundation sizes are the same or smaller (shifted 40 meters for Alternative 3), we have concluded that no Supplemental EIS will be required.
- Even though Alternative 2, with its additional pier, could be perceived as having greater environmental impacts, it would still not meet the criteria for preparing a Supplemental EIS.
- None of the cable-stayed options will require substantial new environmental analyses, and none will result in substantial environmentally related delays.
- Among the three cable-stayed alternatives, Alternatives 1 and 3 would require less follow-up environmental work than Alternative 2, since the latter involves an additional pier in the bay.

Project Delivery

11 Project Delivery

There are a variety of project delivery methods available to Caltrans for the Main Span of the East Span of the San Francisco Oakland Bay Bridge (SFOBB). Included in this list are two that are worthy of discussion in this report: design-bid-build and design-build.

11.1 Design –Bid-Build

The use of design-bid-build is the most common practice in the State of California. It is characterized by the state or its engineering consultant preparing a full set of plans and specifications, soliciting bids from pre-qualified contractors on those plans and specifications and selecting the lowest responsible bidder based on certain selection criteria This selection criteria is generally the lowest price proposed although there are examples where other factors are considered. Once a contractor is selected then the project is built.

The advantages of using design-bid-build is that the owner is very specific in describing what they want the final product to be like and the contractor has a clear understanding of the elements of the project that he will build. The bidding process relies on the contractor providing a price for exactly what is in the plans and specifications.

Design-bid-build is a proven process and was the one that was used on the original bidding of the Self Anchored Suspension (SAS) Bridge in May of 2004. It is a process that is very familiar to both Caltrans and the construction industry in California.

11.2 Design-Build

Design-build has become a very popular delivery method in the last decade. It involves an owner developing a set of plans and specifications that describe the basic attributes of the desired finished product. These plans are often referred to as 15% plans reflecting their general nature and reduced amount of detail. Design-build does not involve the specificity of the design-bid-build process, rather allows the owner to delineate the outcomes and gives the contractor wide latitude to deliver those outcomes in whatever manner they deem appropriate. It is a process that brings significant innovation and creativity to the process of building transportation projects. Over the years, this innovation has brought many very desirable outcomes to owners who have used design-build on their projects. It has also had its share of drawbacks.

Selecting the contractor in a design-build environment can be done using the traditional "low-bid" approach or any number of other selection processes including one that is known as "best value." Best value is a method that allows the owner to consider all of the attributes of a contractor's submittal including price, technical aooroach, quality, timely completion and other factors. In either case, the owner usually ends up with a lump sum price for the whole project.

Design-build has become a very popular means for delivering both complex and relatively simple transportation projects in the United States. It has gained notoriety on large projects such as the I-15 Reconstruction Project in Salt Lake City and the T-Rex

Project in Denver. Both of these projects had a value in excess of $1.5 billion. In addition, many other projects of varied types, such as those in the State of Florida have utilized design-build in very successful ways.

The attributes that are most attractive to users of design-build include fewer changes to the contract once construction is underway, price predictability-meaning that the lump sum price attribute results in fewer cost overruns, more creativity from the contractors and engineers and early completion. Perhaps the greatest and most sought after characteristics are price predictability and early completion.

It is for these two characteristics, price predictability and timely completion that some have suggested that design-build be used for the completion of the Main Span project. For that reason the IRT has determined to add this section to its report. The following observations are offered:

First, for Caltrans to use design-build on the Main Span would require authorizing legislation. Currently, Caltrans is not allowed to use the concept even though many other entities in the state are. Most notably the Self Help Counties have used design-build to advance their projects because they are not limited by statute in its use.

Second, design-build is most effective where there is clarity in the plans and specifications and certainty in the means and methods of the actual construction. In the case of the SAS the plans and specifications remain unclear with many questions from industry remaining to be resolved and the constructibility of the bridge itself in question.

Design-build, with its lump sum price would be desirable for the state in that it would fix, to some degree, the final price of the project. However, given the situation with the plans and specifications and the constructibility issues at hand, the contractors proposing or bidding on an SAS design-build project would likely add significant contingency funds to their price resulting in little if no savings to the state. When the statements about price predictability are made in the design-build world, they are related to well defined and constructible projects—not projects with significant questions and which are ripe for many change orders and contract modifications.

Design-build is a delivery method that requires a different way of doing business on the part of all parties. Contractors and designers take on new and different roles in terms of managing the project, addressing quality issues and in determining solutions that meet the stipulated requirements of the owner. The owner takes on a reduced role in their oversight activities and must be very clear regarding the attributes of the finished product. The good news about design-build is that the owner gets what they ask for. The bad news about design-build is that the owner gets what they ask for. In the end, the owner can and must be very specific about the attributes of their desired finished product and projects with many known and probably unknown issues are not good candidates for design-build.

11.3 Project Delivery Conclusions

Based on the knowledge and experience of the IRT members it is recommended that design-build ***not*** be used for the completion of the Main Span of the SFOBB project if the SAS approach is retained. The reasons are as follows:

1. No authorizing legislation exists to allow Caltrans to use design-build
2. The complexities and anticipated constructibility problems with the SAS design
3. The complexities in dealing with resource agencies as well as local entities
4. The lack of experience in Caltrans in utilizing design-build
5. The lack of procedures and policies within Caltrans to accommodate the use of design-build

This majority of this report by the IRT focuses on the substitution of a cable stayed alternative bridge for the previously designed SAS. It begs the question that if design-build is not a good idea for the SAS approach then would it be appropriate for the cable-stayed alternative? That depends on the following:

- First, can Caltrans get authorization to use design-build from the legislature?
- Second, are the environmental requirements and coordination issues so complicated with resource agencies that design-build would not be advisable?
- Third, is Caltrans prepared as an agency, with their policies and procedures to go forward using design-build?
- Finally, are there any anticipated costs or timesavings associated with using design-build on a cable-stayed alternative?

If the analysis of the project results in affirmative answers to all of these questions then design-build should be considered.

In the end, legislation would be required. Additionally, it is the recommendation that if design-build is utilized for the cable stayed alternative then Caltrans should immediately secure the services of a project management consultant with experience in the development and management of large design-build projects. The IRT does ***not*** recommend advancing design-build on either the SAS or the cable stayed alternative if Caltrans is going to self-manage the project.

Conclusions and Recommendations

12. Conclusions and Recommended

12.1 Background

The Independent Review Team (IRT) has considered volumes of information and inputs while addressing the issues included in this scope of work. These efforts cover the past year and a variety of elements of the TBSRP. It is clear that the analysis shows a cable-stayed option will achieve the schedule and environmental objectives, while providing an equal or better technical solution to the SAS. In addition, the projected savings in excess of $600 million make the redesign option with a cable-stayed bridge a very compelling solution from a fiscal standpoint.

12.2 IRT Conclusions

The results of the additional analysis by the IRT of the advantages, issues, and other factors are summarized in Table 1 for easy reference. The major conclusions from the Phase 2 preliminary design development work are:

1. **Seismic Performance:** The Cable-Stayed alternatives can meet or exceed the seismic design criteria for the SFOBB East Span Project. This includes meeting the strain levels with foundation elements, concrete towers, piers, superstructure, shear link performance and all other elements that govern the seismic performance and safety aspects of the bridge. The concrete towers can be designed to meet the seismic performance requirements of the project. Further information regarding the seismic performance can be found in Sections 3.2.3, 4.2(2), and 6.2(2).
2. **Foundations:** In general, it can be concluded that the foundation sizes and number of piles can remain the same (in some cases the foundations can be smaller) with all of the alternatives. The as-designed SAS foundations can be used for the largest of the Cable-Stayed alternatives (Alternate 1). This assessment is based on similar pile capacity estimates used for the SAS design. However, a review of rock strength data reveals that the pile design used for SAS is extremely conservative. As shown later, the adaptation of a more refined design approach should allow shortening of the drilled shafts at the main tower T1 even for Alternate 1. For other alternates, foundation size can be reduced through redesign, or SAS foundations can be used as is with minor modifications.
3. **Environmental Issues:** The Cable-Stayed design was fully evaluated in the project's Final EIS. Based on the technical analysis performed, the foundation sizes are ***not*** expected to increase for the Cable-Stayed alternatives. There is sufficient reserve capacity in the as-designed SAS foundations at this stage of development that the need to increase their size is hard to comprehend. Further information regarding the foundation capacity can be found in Sections 3.2.3, 4.2(2), and 6.2(2). However, should additional pile capacity be needed for any reason whatsoever, piles can be added within the existing foundation footprints without impacting the foundation sizes.

   Thus the only environmental issues anticipated are: the change of structure type from SAS to Cable-Stayed for all three of the alternatives; the height of the tower above elevation 160.0m for Alternate 1; and the need for one additional foundation in the bay for the Alternate 2. The temporary piers required under the SAS design would be eliminated under the Cable-Stayed alternatives.

Both the SAS and cable-stayed designs were fully evaluated as design options under the Preferred Alternative in the SFOBB's Final Environmental Impact Statement (FEIS) that was completed in 2001. The FEIS concluded that the overall environmental impacts of these two options were virtually identical. ***All necessary environmental work can be accomplished through a reevaluation process with minor modifications to existing permits as necessary.*** Additional environmental documentation and modification of existing permits for the Cable-Stayed alternatives can be accomplished in a 9-month period.

4. **Impacts to YBI and Skyway Interfaces:** In general, all of the options considered had little or no impact to the YBI interface. In any case, if some change is needed to the YBI interface, it can be incorporated into the design as it is still under development. On the Skyway side, some of the schemes (for example, Alternate 1, transition option A) have no impact to the interface where as other schemes would have some resolvable design issues. These would simply be designed into the interface and appropriate changes made to the skyway contract.

5. **Cost Savings:** The estimated net cost savings for Alternates 1 and 3 exceed $600 million. Further, there is an additional estimated savings in excess of $250 million for potential additional costs during construction as the Cable-Stayed design is judged to have less risk with respect to its fabrication and erection. The same can be inferred for Alternate 2. These cost savings are based on the assumed base price of $1.58 billion ($ 1.4 billion on SAS recent bid and $178 million on E2/T1).

6. **Schedule Impacts:** All of the Cable-Stayed alternatives can be constructed by or before the theoretical SAS construction timeline. However, if construction were to proceed on the SAS design, there are overwhelming reasons to expect significant schedule creep during construction, thus all of the Cable-Stayed alternatives provide significant schedule advantages over SAS. Detailed schedules were developed for the Cable-Stayed alternates in two scenarios. The first scenario assumed no redesign (except for some minor potential adjustments) of the foundations and the second scenario assumed that the foundations would be significantly redesigned. The detailed schedules developed for the different alternates under these two scenarios are given in Section 7. The feasibility of the use of existing SAS foundations provides schedule advantages in addition to the direct economic advantages.

7. **SAS Risks:** One of the elements of the SAS Bridge that the IRT was asked to review concerned the risk characteristics associated with the construction of the SAS. The single tower SAS of this size and constructed in this environment is a first-of-a-kind bridge. Even though a bid had been received there is no reasonable assurance that it could be built within the bid price and schedule. Section 9 details numerous risks associated with constructing the SAS. These risks could add several years to the schedule for completing the SAS design. In addition, it is recommended to budget a construction contingency of $350,000,000 to address these items if the SAS design is pursued. Experience indicates that first-of-a-kind major bridges have a high potential for construction claims, added costs and schedule delays.

8. **Project Delivery Method:** There are two primary project delivery methods: Design-Bid-Build and Design-Build. Based on the knowledge and experience of the IRT members, it is recommended that design-build **not** be used for the completion of the Main Span of the SFOBB project if the SAS approach is retained. This is largely due to the complexity of the SAS design and inexperience of Caltrans in utilizing design-build, especially on such a complex project.

   Design-build could be considered with a cable-stayed alternative as there is not the level of complexity, uncertainty and inexperience with the cable-stayed design as there is with the SAS. Design-build could be considered for the cable-stayed design if the following conditions were met.

- Obtain authorization to use design-build from the legislature
- Validate that the environmental requirements and coordination issues with resource agencies will not be a detriment to the design-build process
- Prepare Caltrans with the policies and procedures to go forward using design-build
- Validate that there are costs or time savings associated with using design-build on a cable-stayed alternative

If the analysis of the project results in affirmative answers to all of these questions then design-build should be considered. Additionally, it is the recommendation of the IRT that if design-build is utilized for the Cable-Stayed alternative then Caltrans should immediately secure the services of a project management consultant with experience in the development and management of large design-build projects. The IRT does ***not*** recommend advancing design-build on either the SAS or the Cable-Stayed alternative if the project is going to be self-managed by Caltrans.

12.3 IRT Recommendations

Based on the findings from our study, the IRT recommends proceeding with the redesign of a selected Cable-Stayed alternate. As there are significant cost impacts associated with delays to the current E2/T1 foundation contract, time is of the essence. Alternate 1 offers the most advantages with respect to schedule and Alternate 3 offers the most in estimated cost savings. Alternate 2 requires evaluation of an additional foundation in the bay, which has potential for schedule delay and offers no real advantage over Alternate 1 or 3.

The IRT offers the following recommendations for the State of California:

1. Immediately adopt the redesign option and select either Cable-Stayed Alternative 1 or 3 as the course of action for moving forward on the main span of the SFOBB.
2. Immediately procure the services of an engineering consulting firm to complete the design work related to the Cable-Stayed option selected in #1 above.
3. Immediately complete a detailed cost analysis for the Cable-Stayed option selected for inclusion in the program budget for the TBSRP for presentation to the legislature.
4. Immediately develop a course of action to deal with the current E2/T1 contract under construction by Kiewit.
5. Immediately start the environmental reevaluation process and any necessary permit modifications.

Appendix A

APPENDIX A – Detailed Cost Estimates

Appendix A provides construction cost estimates for both Alternate 1 and Alternate 3 escalated to midpoint of construction. These estimates were prepared in contractor methodology by Peter Sanderson, a construction expert with significant experience bidding and building long span structures including bridges in California. The estimates are based on updated quantities from the recent seismic analysis studies.

In addition, the miscellaneous items from the Bid Analysis are included as these values are not included in Mr. Sanderson's estimates and needed to be added to the total Cable-Stayed estimates for comparison.

PETER SANDERSON
655 BROADWAY, SUITE 800,
DENVER, COLORADO, 80203

303 886 6679
604 244 7343
604 244 7340 FAX

November 1, 2004

Fax only: Sena Kumarasena 617 428 6905
Ray McCabe 212 594 9638

Re: SFOBB Cable Stayed Alternatives.

The attached ten pages summarize my re-estimates for these two alternatives.

Estimate 0416 is the 180/385 Alternative, which totals out at $535,845,411

Estimate 0417 is the 140/385/140 Alternative , total being ~~$496,030,303~~

$432,358,178

Revised 11/16/04 to remove foundation items at T1 & E2. These are separately included in the cost sheet as a $125 M item.

Peter Sanderson Consulting, Inc.
04016 SFOBB CABLE STAYED 180/385 B&S
ult User

Page 1
11/01/2004 14:01

ESTIMATE SUMMARY (BID PRICES)

Bid #	Quantity	Client Bid# Unit		Total Cost	Total Cost Unit Price	Markup	Balanced Bid Total	Balanced Bid Unit Price	Pricing Status	Bid Price	Bid Total
920	1.00	LS	Erection Manual	316,228.37	316,228.37	0.0% 0	316,228	316,228.37		316,228.370	316,228.37
922	1.00	LS	Step by Step Field Instructions	632,456.75	632,456.75	18.9% 119,725	752,181	752,181.49		752,181.490	752,181.49
923	1.00	LS	Construction Engineering	3,952,854.68	3,952,854.68	18.9% 748,280	4,701,134	4,701,134.29		4,701,134.290	4,701,134.29
927	1.00	LS	As Built Drawing Preparation	191,542.37	191,542.37	18.9% 36,259	227,802	227,801.55		227,801.550	227,801.55
1000	1.00	LS	Cost to Complete Review	118,585.64	118,585.64	18.9% 22,448	141,034	141,034.03		141,034.030	141,034.03
1020	1.50	LS	Access Trestle to Tower	2,775,258.93	1,850,172.62	18.9% 525,359	3,300,618	2,200,412.27		2,200,412.270	3,300,618.41
1030	1.00	LS	Access Roads & Bridges at Site	631,854.52	631,854.52	18.9% 119,611	751,465	751,465.26		751,465.260	751,465.26
1040	1.00	LS	Excavation for W2	2,886,849.93	2,886,849.93	18.9% 546,484	3,433,334	3,433,333.71		3,433,333.710	3,433,333.71
1050	1.00	LS	Shoring for W2	589,768.37	589,768.37	18.9% 111,644	701,412	701,412.15		701,412.150	701,412.15
1060	1.00	LS	Backfill	475,110.85	475,110.85	18.9% 89,939	565,050	565,049.84		565,049.840	565,049.84
2020	6,387.00	M3	Foundation Concrete for W2	5,080,798.09	795.49	18.9% 961,801	6,042,599	946.08		946.080	6,042,612.96
2021	1,900,610.00	KG	Foundation Reinforcing for W2	3,660,604.07	1.93	18.9% 692,956	4,353,560	2.29		2.290	4,352,396.90
2030	3,194.00	M3	Pier Concrete for W2	11,130,334.04	3,484.76	18.9% 2,106,984	13,237,318	4,144.43		4,144.430	13,237,309.42
2040	2,617,502.00	KG	Pier Concrete Reinforcement for W2	13,025,057.80	4.98	18.9% 2,465,657	15,490,715	5.92		5.920	15,495,611.84
2052	177,944.00	KG	Tie Downs In W2	3,102,178.44	17.43	18.9% 587,246	3,689,424	20.73		20.730	3,688,779.12
53	4.00	EA	Bearings at W2	778,936.36	194,734.09	18.9% 147,453	926,390	231,597.46		231,597.460	926,389.84

Bid #	Quantity	Client Bid# Unit	Description	Total Cost	Total Cost Unit Price	Markup %	Markup	Balanced Bid Total	Balanced Bid Unit Price	Pricing Status	Bid Price	Bid Total
2054	58.00	M	Expansion Joint at W2	336,593.47	5,803.34	18.9%	63,718	400,311	6,901.91		6,901.910	400,310.78
2072	1,000.00	M3	Cross Beam Concrete for E2	1,317,974.39	1,317.97	18.9%	249,494	1,567,468	1,567.47		1,567.470	1,567,470.00
2073	150,000.00	M3	Cross Beam Concrete Reinforcement	666,483.10	4.44	18.9%	126,166	792,649	5.28		5.280	792,000.00
2074	26,438.00	KG	Tie Downs in E2	506,833.46	19.17	18.9%	95,944	602,778	22.80		22.800	602,786.40
2075	1.00	LS	Tower Foundations			0.0%	0	0				
2080	13,064.00	M3	Concrete for Tower	38,497,830.46	2,946.86	18.9%	7,287,680	45,785,511	3,504.71		3,504.710	45,785,531.44
2081	6,160,591.00	KG	Concrete Reinforcement For Tower	29,962,557.91	4.86	18.9%	5,671,944	35,634,502	5.78		5.780	35,608,215.98
2082	30,000.00	KG	Concrete Post Tensioning For Tower	304,380.04	10.15	18.9%	57,619	362,000	12.07		12.070	362,100.00
2083	1,000,000.00	KG	Struct & Misc Steel For Tower	26,635,096.83	26.64	18.9%	5,042,052	31,677,149	31.68		31.680	31,680,000.00
3000	2,700,000.00	KG	Stay Cables	56,351,745.72	20.87	18.9%	10,667,446	67,019,191	24.82		24.820	67,014,000.00
3100	16,143,564.00	KG	Structural Steel for Mainspan	153,341,251.61	9.50	18.9%	29,027,663	182,368,914	11.30		11.300	182,422,273.20
3110	1,275,000.00	KG	Bike Path Steel	10,953,302.46	8.59	18.9%	2,073,472	13,026,774	10.22		10.220	13,030,500.00
3111	32,422.00	SF	Bike Path Topping	410,110.25	12.65	18.9%	77,634	487,745	15.04		15.040	487,626.88
3200	7,560.00	M3	Precast Concrete for Mainspan	18,589,487.25	2,458.93	18.9%	3,519,010	22,108,497	2,924.40		2,924.400	22,108,464.00
3300	1,072.00	M3	Cast in Place Concrete for Mainspa	1,351,416.69	1,260.65	18.9%	255,825	1,607,241	1,499.29		1,499.290	1,607,238.88
[illegible]1	252,870.00	KG	Concrete Reinforcement for Mainspa	1,786,044.67	7.06	18.9%	338,100	2,124,145	8.40		8.400	2,124,108.00

Peter Sanderson Consulting, Inc.
04016 SFOBB CABLE STAYED 180/385 B&S
[illegible]ault User

ESTIMATE SUMMARY (BID PRICES)

Bid #	Quantity	Client Bid# Unit	Description	Total Cost	Total Cost Unit Price	Markup	Markup	Balanced Bid Total	Balanced Bid Unit Price	Pricing Status	Bid Price	Bid Total
3302	360,656.00	KG	Transverse PT For deck	4,647,563.76	12.89	18.9%	879,789	5,527,353	15.33		15.330	5,528,856.48
3400	35,130.00	M2	Wearing Surface	2,656,318.34	75.61	18.9%	502,844	3,159,162	89.93		89.930	3,159,240.90
3500	1,240.00	M	Bicycle Railing	625,702.00	504.60	18.9%	118,446	744,148	600.12		600.120	744,148.80
3501	2,480.00	M	K Rail	705,821.73	284.61	18.9%	133,613	839,435	338.48		338.480	839,430.40
3502	2,480.00	M	Cable Railing B11	310,284.62	125.11	18.9%	58,737	369,022	148.80		148.800	369,024.00
3503	1,530.00	M	Handrail S11	244,960.55	160.10	18.9%	46,371	291,332	190.41		190.410	291,327.30
3600	24.00	EA	Bearings & Restrainers	3,550,315.10	147,929.80	18.9%	672,078	4,222,394	175,933.06		175,933.060	4,222,393.44
3610	219.00	M	Expansion & Breather Joints	1,430,781.75	6,533.25	18.9%	270,849	1,701,630	7,770.00		7,770.000	1,701,630.00
4000	7,088.00	M3	Cast In Place Concrete Backspan	20,519,192.18	2,894.92	18.9%	3,884,305	24,403,497	3,442.93		3,442.930	24,403,487.84
4001	1.00	LS	Falsework to water level	1,850,164.75	1,850,164.75	18.9%	350,238	2,200,403	2,200,402.91		2,200,402.910	2,200,402.91
4002	360.00	M	Falsework above water and land	9,505,711.05	26,404.75	18.9%	1,799,441	11,305,152	31,403.20		31,403.200	11,305,152.00
4010	566,356.00	KG	CIP Concrete Backspan- Reinforcing	3,254,042.62	5.75	18.9%	615,994	3,870,036	6.83		6.830	3,868,211.48
4020	120,100.00	KG	CIP Concrete Backspan- Post Tensio	1,403,933.75	11.69	18.9%	265,766	1,669,700	13.90		13.900	1,669,390.00
4030	100,000.00	KG	CIP Concrete Backspan/E2-Misc Met	1,368,500.44	13.69	18.9%	259,059	1,627,559	16.28		16.280	1,628,000.00
4061	21,800.00	KG	E2 Segmental Closure Skyway Reinfo	257,019.40	11.79	18.9%	48,654	305,673	14.02		14.020	305,636.00
[illegible]63	109.00	M3	E2 Segmental Closure Mainspan	746,334.24	6,847.10	18.9%	141,282	887,616	8,143.27		8,143.270	887,616.43

Peter Sanderson Consulting, Inc.
04016 SFOBB CABLE STAYED 180/385 B&S
ult User

Page 4
11/01/2004 14:01

ESTIMATE SUMMARY (BID PRICES)

Bid #	Quantity	Client Bid# Unit	Total Cost	Total Cost Unit Price	Markup	Balanced Bid Total	Balanced Bid Unit Price	Pricing Status	Bid Price	Bid Total
4064			E2 Segmental Closure Mainspan Rein		18.9%					
	21,800.00	KG	257,019.40	11.79	48,654	305,673	14.02		14.020	305,636.00
5000			Bridge Electrical/Lighting		18.9%					
	1.00	LS	3,162,283.74	3,162,283.74	598,624	3,760,907	3,760,907.43		3,760,907.430	3,760,907.43
5025			Mass Concrete & Concrete Cooling		18.9%					
	1.00	LS	1,411,434.83	1,411,434.83	267,186	1,678,621	1,678,620.95		1,678,620.950	1,678,620.95
5030			Stain Finish		18.9%					
	1.00	LS	1,413,540.83	1,413,540.83	267,585	1,681,126	1,681,125.62		1,681,125.620	1,681,125.62
7050			Bridge Drains		18.9%					
	1.00	LS	158,114.19	158,114.19	29,931	188,045	188,045.37		188,045.370	188,045.37
7060			Column Inspection System		18.9%					
	1.00	LS	31,622.84	31,622.84	5,986	37,609	37,609.08		37,609.080	37,609.08
7070			Crash Cushions- Median		18.9%					
	1.00	LS	126,491.35	126,491.35	23,945	150,436	150,436.30		150,436.300	150,436.30
.080			Light Standard Bases		18.9%					
	1.00	LS	79,057.09	79,057.09	14,966	94,023	94,022.68		94,022.680	94,022.68
7090			Sign Bases		18.9%					
	1.00	LS	79,057.09	79,057.09	14,966	94,023	94,022.68		94,022.680	94,022.68
7100			Miscellaneous Metals-Hatches etc		18.9%					
	1.00	LS	79,057.09	79,057.09	14,966	94,023	94,022.68		94,022.680	94,022.68
7500			Deck Grinding		18.9%					
	1.00	LS	345,605.99	345,605.99	65,424	411,030	411,029.57		411,029.570	411,029.57
		TOTALS:	450,579,458.24		85,235,310	535,814,769				535,845,411.08

Code between Balanced Bid & Bid Price: U-Unbalanced, F-Frozen, C-Closing Biditem (item to absorb unbalancing differences).

** in front of the Biditem indicates a Non-Additive item

ESTIMATE SUMMARY (BID PRICES)

Bid # Quantity	Client Bid# Unit	Total Cost	Total Cost Unit Price	Markup	Balanced Bid Total	Balanced Bid Unit Price	Pricing Status	Bid Price	Bid Total

BID COSTS ===> 450,579,458
ACTUAL MARKUP ===> 85,265,953
TOTAL BID ===> 535,845,411

Spread Indirects on: TOTAL COST Spread Markup on: TOTAL COST Spread Addons&Bond on: TOTAL COST

Default Markup %:

Labor:	50%
Burden:	50%
Perm Matl:	0%
Const Matl:	20%
Sub:	0%
Eqp Op Exp:	20%
Co. Equip:	20%
Rented Eqp:	20%
Misc 1:	0%
Misc 2:	0%
Misc 3:	0%

——— ESTIMATE NOTES: ———

Bid Date: 06/12/2004 Owner: Engineering Firm:
Estimator-In-Charge: PFS Desired Bid (if specified) = 0

Last Summary on 11/01/2004 at 1:59 PM.
Last Spread on 11/01/2004 at 1:59 PM.

Peter Sanderson Consulting, Inc.
04017 SFOBB 140/385/140
[illegible]lt User

Page 1
11/16/2004 13:57

ESTIMATE SUMMARY (BID PRICES)

Bid #	Quantity	Client Bid# Unit	Total Cost	Total Cost Unit Price	Markup	Balanced Bid Total	Balanced Bid Unit Price	Pricing Status	Bid Price	Bid Total
920			Erection Manual		0.0%					
	1.00	LS	324,329.29	324,329.29	0	324,329	324,329.29		324,329.290	324,329.29
922			Step by Step Field Instructions		18.6%					
	1.00	LS	648,658.59	648,658.59	120,369	769,028	769,027.63		769,027.630	769,027.63
923			Construction Engineering		18.6%					
	1.00	LS	4,054,116.17	4,054,116.17	752,307	4,806,423	4,806,422.67		4,806,422.670	4,806,422.67
927			As Built Drawing Preparation		18.6%					
	1.00	LS	196,449.17	196,449.17	36,454	232,903	232,903.48		232,903.480	232,903.48
1000			Cost to Complete Review		18.6%					
	1.00	LS	121,623.49	121,623.49	22,569	144,193	144,192.68		144,192.680	144,192.68
1020			Access Trestle to Tower		18.6%					
	0.50	LS	948,792.95	1,897,585.90	176,064	1,124,857	2,249,713.50		2,249,713.500	1,124,856.75
1030			Access Roads & Bridges at Site		18.6%					
	1.00	LS	648,040.94	648,040.94	120,254	768,295	768,295.36		768,295.360	768,295.36
2050			Cross Beam Concrete for W2		18.6%					
	700.00	M3	726,747.87	1,038.21	134,860	861,608	1,230.87		1,230.870	861,609.00
2051			Cross Beam Reinforcement for W2		18.6%					
	210,000.00	KG	750,282.47	3.57	139,227	889,509	4.24		4.240	890,400.00
2052			Tie Downs In W2		18.6%					
	100,000.00	KG	1,788,000.25	17.88	331,792	2,119,792	21.20		21.200	2,120,000.00
2053			Bearings at W2		18.6%					
	4.00	EA	798,890.61	199,722.65	148,247	947,138	236,784.41		236,784.410	947,137.64
2054			Expansion Joint at W2		18.6%					
	58.00	M	345,216.08	5,952.00	64,060	409,276	7,056.49		7,056.490	409,276.42
3000			Stay Cables		18.6%					
	1,557,402.00	KG	32,356,360.32	20.78	6,004,243	38,360,604	24.63		24.630	38,358,811.26
3100			Structural Steel for Mainspan		18.6%					
	17,662,597.00	KG	170,533,788.87	9.66	31,645,289	202,179,078	11.45		11.450	202,236,735.65
3111			Bike Path Topping		18.6%					
	34,910.00	SF	452,893.43	12.97	84,042	536,935	15.38		15.380	536,915.80
3200			Precast Concrete for Mainspan		18.6%					
	8,242.00	M3	27,810,620.42	3,374.26	5,160,708	32,971,329	4,000.40		4,000.400	32,971,296.80

Peter Sanderson Consulting, Inc.
04017 SFOBB 140/385/140
lt User

Page 2
11/16/2004 13:57

ESTIMATE SUMMARY (BID PRICES)

Bid #	Quantity	Client Bid# / Unit	Total Cost	Total Cost Unit Price	Markup	Balanced Bid Total	Balanced Bid Unit Price	Pricing Status	Bid Price	Bid Total
3300	1,169.00	M3	Cast in Place Concrete for Mainspa 1,511,450.42	1,292.94	18.6% 280,474	1,791,924	1,532.87		1,532.870	1,791,925.03
3301	275,670.00	KG	Concrete Reinforcement for Mainspa 1,996,959.50	7.24	18.6% 370,568	2,367,527	8.59		8.590	2,368,005.30
3302	400,000.00	KG	Transverse PT For deck 5,286,611.44	13.22	18.6% 981,016	6,267,627	15.67		15.670	6,268,000.00
3400	39,900.00	M2	Wearing Surface 3,094,284.39	77.55	18.6% 574,194	3,668,479	91.94		91.940	3,668,406.00
3500	1,330.00	M	Bicycle Railing 688,312.71	517.53	18.6% 127,728	816,040	613.56		613.560	816,034.80
3501	2,660.00	M	K Rail 776,444.33	291.90	18.6% 144,082	920,526	346.06		346.060	920,519.60
3502	2,660.00	M	Cable Railing B11 341,329.50	128.32	18.6% 63,339	404,669	152.13		152.130	404,665.80
3503	1,530.00	M	Handrail S11 251,235.78	164.21	18.6% 46,621	297,857	194.68		194.680	297,860.40
3600	32.00	EA	Bearings & Restrainers 4,855,016.42	151,719.26	18.6% 900,926	5,755,943	179,873.21		179,873.210	5,755,942.72
3610	219.00	M	Expansion & Breather Joints 1,467,434.52	6,700.61	18.6% 272,306	1,739,741	7,944.02		7,944.020	1,739,740.38
4060	109.00	M3	E2 Segmental Closure Skyway 765,453.31	7,022.51	18.6% 142,042	907,495	8,325.65		8,325.650	907,495.85
4061	21,800.00	KG	E2 Segmental Closure Skyway Reinfo 263,603.55	12.09	18.6% 48,916	312,519	14.34		14.340	312,612.00
4063	109.00	M3	E2 Segmental Closure Mainspan 765,453.31	7,022.51	18.6% 142,042	907,495	8,325.65		8,325.650	907,495.85
4064	21,800.00	KG	E2 Segmental Closure Mainspan Rein 263,603.55	12.09	18.6% 48,916	312,519	14.34		14.340	312,612.00
4078	1,120.00	M3	Pedestral Concrete 913,774.19	815.87	18.6% 169,566	1,083,340	967.27		967.270	1,083,342.40
4079	15,363.00	M3	Tower Wall Concrete 47,055,607.85	3,062.92	18.6% 8,731,925	55,787,533	3,631.29		3,631.290	55,787,508.27

Peter Sanderson Consulting, Inc. Page 3
04017 SFOBB 140/385/140 11/16/2004 13:57
Default User

ESTIMATE SUMMARY (BID PRICES)

Bid # / Quantity	Client Bid# / Unit	Total Cost	Total Cost Unit Price	Markup	Balanced Bid Total	Balanced Bid Unit Price	Pricing Status	Bid Price	Bid Total
4080		Cross Beam Concrete		18.6%					
3,500.00	M3	3,633,739.32	1,038.21	674,299	4,308,038	1,230.87		1,230.870	4,308,045.00
4081		Tower Concrete Rebar		18.6%					
7,247,041.00	KG	36,149,506.20	4.99	6,708,123	42,857,629	5.91		5.910	42,830,012.31
4082		Tower Post Tensioning		18.6%					
300,000.00	KG	3,121,698.43	10.41	579,281	3,700,980	12.34		12.340	3,702,000.00
4083		Tower Misc Metals		18.6%					
100,000.00	KG	2,731,742.97	27.32	506,919	3,238,662	32.39		32.390	3,239,000.00
5000		Bridge Electrical/Lighting		18.6%					
1.00	LS	3,243,292.94	3,243,292.94	601,845	3,845,138	3,845,138.14		3,845,138.140	3,845,138.14
5025		Mass Concrete & Concrete Cooling		18.6%					
1.00	LS	1,447,591.99	1,447,591.99	268,624	1,716,216	1,716,215.98		1,716,215.980	1,716,215.98
5030		Stain Finish		18.6%					
1.00	LS	1,449,751.94	1,449,751.94	269,025	1,718,777	1,718,776.74		1,718,776.740	1,718,776.74
7050		Bridge Drains		18.6%					
1.00	LS	162,164.65	162,164.65	30,092	192,257	192,256.91		192,256.910	192,256.91
7060		Column Inspection System		18.6%					
1.00	LS	32,432.93	32,432.93	6,018	38,451	38,451.38		38,451.380	38,451.38
7070		Crash Cushions- Median		18.6%					
1.00	LS	129,731.72	129,731.72	24,074	153,806	153,805.53		153,805.530	153,805.53
7080		Light Standard Bases		18.6%					
1.00	LS	81,082.32	81,082.32	15,046	96,128	96,128.45		96,128.450	96,128.45
7090		Sign Bases		18.6%					
1.00	LS	81,082.32	81,082.32	15,046	96,128	96,128.45		96,128.450	96,128.45
7100		Miscellaneous Metals-Hatches etc		18.6%					
1.00	LS	81,082.32	81,082.32	15,046	96,128	96,128.45		96,128.450	96,128.45
7500		Deck Grinding		18.6%					
1.00	LS	354,459.49	354,459.49	65,776	420,235	420,235.15		420,235.150	420,235.15
9000		Salvage of Equipment & Materials		18.6%					
1.00	LS	-791,622.94	-791,622.94	-146,897	-938,520	-938,521.32		-938,521.320	-938,521.32
TOTALS:		364,709,122.29		67,617,462	432,326,585				432,358,178.00

Peter Sanderson Consulting, Inc. Page 5
04017 SFOBB 140/385/140 11/16/2004 13:57
Default User

ESTIMATE SUMMARY (BID PRICES)

Bid # Quantity	Client Bid# Unit	Total Cost	Total Cost Unit Price	Markup	Balanced Bid Total	Balanced Bid Unit Price	Pricing Status	Bid Price	Bid Total

BID COSTS ==========>	364,709,122
ACTUAL MARKUP =====>	67,649,056
TOTAL BID ==========>	432,358,178

Spread Indirects on: TOTAL COST Spread Markup on: TOTAL COST Spread Addons&Bond on: TOTAL COST

Default Markup %:

Labor:	50%
Burden:	50%
Perm Matl:	0%
Const Matl:	20%
Sub:	0%
Eqp Op Exp:	20%
Co. Equip:	20%
Rented Eqp:	20%
Misc 1:	0%
Misc 2:	0%
Misc 3:	0%

------ ESTIMATE NOTES: ------

Bid Date: 06/12/2004 Owner: Engineering Firm:
Estimator-In-Charge: PFS Desired Bid (if specified) = 0

Last Summary on 11/16/2004 at 1:46 PM.
Last Spread on 11/16/2004 at 1:46 PM.

04-0120F1 BID ANALYSIS 052604

ITEM NUMBER	ITEM CODE	DESCRIPTION	UNIT	QUANTITIES 100% 4/27/04 BEES	UNIT PRICE 4/27/04 BEES	UNIT PRICE PROBABLE BID	UNIT PRICES-BIDDER #1 INTERNATIONAL	UNIT PRICES-BIDDER #1 DOMESTIC	AMOUNT 4/27/04 BEES	AMOUNT PROBABLE BID	BID AMOUNT- BIDDER #1 INTERNATIONAL	BID AMOUNT- BIDDER #1 DOMESTIC	AMOUNT OVER (UNDER)		AMOUNT OVER (UNDER)	
1	030748	WORKING DRAWING CAMPUS	LS	1	5,000,000.00	5,000,000.00	5,000,000.00	5,000,000.00	5,000,000.00	5,000,000.00	5,000,000.00	5,000,000.00	-	-	-	-
2	030702	ELECTRONIC MOBILE DAILY DIARY COMPUTER	LS	1	24,054.90	24,054.90	20,000.00	20,000.00	24,054.90	24,054.90	20,000.00	20,000.00	(4,054.90)	-17%	(4,054.90)	-17%
3	000001	~~PILE CORROSION MONTIORING SYSTEM~~ DELETED PER ADDENDUM #4	LS	1	-	-	-	-	-	-	-	-	-	-	-	-
4	030704	EROSION CONTROL (TYPE B)	M2	1,570	25.00	25.00	25.00	25.00	39,250.00	39,250.00	39,250.00	39,250.00	-	-	-	-
5	070010	PROGRESS SCHEDULE (CRITICAL PATH)	LS	1	100,000.00	100,000.00	100,000.00	100,000.00	100,000.00	100,000.00	100,000.00	100,000.00	-	-	-	-
7	071322	TEMPORARY FENCE (TYPE CL-1.8)	M	205	38.00	38.00	30.00	30.00	7,790.00	7,790.00	6,150.00	6,150.00	(1,640.00)	-21%	(1,640.00)	-21%
8	030705	3.66 M TEMPORARY GATE (TYPE CL-1.8)	EA	1	800.00	800.00	2,000.00	2,000.00	800.00	800.00	2,000.00	2,000.00	1,200.00	150%	1,200.00	150%
27	074019	PREPARE STORM WATER POLLUTION PREVENTION PLAN	LS	1	10,000.00	10,000.00	50,000.00	50,000.00	10,000.00	10,000.00	50,000.00	50,000.00	40,000.00	400%	40,000.00	400%
28	074020	WATER POLLUTION CONTROL	LS	1	110,000.00	110,000.00	100,000.00	100,000.00	110,000.00	110,000.00	100,000.00	100,000.00	(10,000.00)	-9%	(10,000.00)	-9%
29	030706	NON-STORM WATER DISCHARGES	LS	1	60,000.00	60,000.00	100,000.00	100,000.00	60,000.00	60,000.00	100,000.00	100,000.00	40,000.00	67%	40,000.00	67%
30	030632	TURBIDITY CONTROL	LS	1	40,000.00	40,000.00	100,000.00	100,000.00	40,000.00	40,000.00	100,000.00	100,000.00	60,000.00	150%	60,000.00	150%
31	074032	TEMPORARY CONCRETE WASHOUT FACILITY	LS	1	60,000.00	60,000.00	100,000.00	100,000.00	60,000.00	60,000.00	100,000.00	100,000.00	40,000.00	67%	40,000.00	67%
32	074034	TEMPORARY COVER	M2	1,350	7.00	7.00	15.00	15.00	9,450.00	9,450.00	20,250.00	20,250.00	10,800.00	114%	10,800.00	114%
34	120100	TRAFFIC CONTROL SYSTEM	LS	1	240,000.00	240,000.00	50,000.00	50,000.00	240,000.00	240,000.00	50,000.00	50,000.00	(190,000.00)	-79%	(190,000.00)	-79%
35	150605	REMOVE FENCE	M	90	17.30	17.30	20.00	20.00	1,557.00	1,557.00	1,800.00	1,800.00	243.00	16%	243.00	16%
36	150620	REMOVE GATE	EA	2	289.00	289.00	500.00	500.00	578.00	578.00	1,000.00	1,000.00	422.00	73%	422.00	73%
37	030709	RECONSTRUCT CHAIN LINK FENCE (TYPE CL-2.4 BLACK VINYL-CLAD)WITH BARBED WIRED EXTENSION ARMS	M	150	150.00	150.00	30.00	30.00	22,500.00	22,500.00	4,500.00	4,500.00	(18,000.00)	-80%	(18,000.00)	-80%
38	030710	RECONSTRUCT CHAIN LINK GATE (TYPE CL-2.4 BLACK VINYL-CLAD)WITH BARBED WIRED EXTENSION ARMS	EA	2	1,550.00	1,550.00	1,000.00	1,000.00	3,100.00	3,100.00	2,000.00	2,000.00	(1,100.00)	-35%	(1,100.00)	-35%
39	032138	~~STRAW (EROSION CONTROL)~~ STABILILIZING EMULSION	KG	35	5.30	5.30	5.00	5.00	185.50	185.50	175.00	175.00	(10.50)	-6%	(10.50)	-6%
40	203014	FIBER (EROSION CONTROL)	KG	155	1.50	1.50	5.00	5.00	232.50	232.50	775.00	775.00	542.50	233%	542.50	233%
41	203021	FIBER ROLLS	M	252	14.50	14.50	20.00	20.00	3,654.00	3,654.00	5,040.00	5,040.00	1,386.00	38%	1,386.00	38%
42	203024	COMPOST EROSION CONTROL	KG	470	1.50	1.50	1.00	1.00	705.00	705.00	470.00	470.00	(235.00)	-33%	(235.00)	-33%
43	030711	MOVE IN/OUT (EROSION CONTROL)	EA	4	650.00	650.00	2,000.00	2,000.00	2,600.00	2,600.00	8,000.00	8,000.00	5,400.00	208%	5,400.00	208%
44	203045	PURE LIVE SEED (EROSION CONTROL)	KG	30	65.00	65.00	100.00	100.00	1,950.00	1,950.00	3,000.00	3,000.00	1,050.00	54%	1,050.00	54%
54	049307	STRUCTURAL CONCRETE, FENDER	M3	1,204	800.00	800.00	3,000.00	3,000.00	963,200.00	963,200.00	3,612,000.00	3,612,000.00	2,648,800.00	275%	2,648,800.00	275%
89	030712	SERVICE PLATFORM	EA	5	242,600.00	242,600.00	60,000.00	60,000.00	1,213,000.00	1,213,000.00	300,000.00	300,000.00	(913,000.00)	-75%	(913,000.00)	-75%
90	560218	FURNISH SIGN STRUCTURE (TRUSS)	KG	9,200	4.80	4.80	6.00	6.00	44,160.00	44,160.00	55,200.00	55,200.00	11,040.00	25%	11,040.00	25%
91	560219	ERECT SIGN STRUCTURE (TRUSS)	KG	9,200	1.50	1.50	1.00	1.00	13,800.00	13,800.00	9,200.00	9,200.00	(4,600.00)	-33%	(4,600.00)	-33%
92	562002	METAL (BARRIER MOUNTED SIGN)	KG	1,020	15.60	15.60	15.00	15.00	15,912.00	15,912.00	15,300.00	15,300.00	(612.00)	-4%	(612.00)	-4%
93	049333	PLASTIC LUMBER	M3	99	2,840.00	2,840.00	5,000.00	5,000.00	281,160.00	281,160.00	495,000.00	495,000.00	213,840.00	76%	213,840.00	76%
94	049334	UHMW POLYETHYLENE PANEL (50 MM)	M2	637	310.00	310.00	1,000.00	1,000.00	197,470.00	197,470.00	637,000.00	637,000.00	439,530.00	223%	439,530.00	223%
102	030713	PERIMETER FENCE (TYPE WM 1.8)	M	410	18.00	18.00	70.00	70.00	7,380.00	7,380.00	28,700.00	28,700.00	21,320.00	289%	21,320.00	289%
103	833020	CHAIN LINK RAILING	M	130	80.00	80.00	150.00	150.00	10,400.00	10,400.00	19,500.00	19,500.00	9,100.00	88%	9,100.00	88%
108	840515	THERMOPLASTIC PAVEMENT MARKING	M2	18	49.90	49.90	80.00	80.00	898.20	898.20	1,440.00	1,440.00	541.80	60%	541.80	60%
109	840561	100 MM THERMOPLASTIC TRAFFIC STRIPE	M	7,500	1.90	1.90	2.00	2.00	14,250.00	14,250.00	15,000.00	15,000.00	750.00	5%	750.00	5%
110	030715	75 MM PAINT TRAFFIC STRIPE (BLACK, 1-COAT)	M	2,500	1.20	1.20	2.00	2.00	3,000.00	3,000.00	5,000.00	5,000.00	2,000.00	67%	2,000.00	67%
111	840656	PAINT TRAFFIC STRIPE (2-COAT)	M	590	1.90	1.90	2.00	2.00	1,121.00	1,121.00	1,180.00	1,180.00	59.00	5%	59.00	5%
112	840666	PAINT PAVEMENT MARKING (2-COAT)	M	8	69.30	69.30	100.00	100.00	554.40	554.40	800.00	800.00	245.60	44%	245.60	44%
113	850101	PAVEMENT MARKER (NON-REFLECTIVE)	M	1,390	2.00	2.00	4.00	4.00	2,780.00	2,780.00	5,560.00	5,560.00	2,780.00	100%	2,780.00	100%
114	850111	PAVEMENT MARKER (RETROREFLECTIVE)	M	440	3.00	3.00	6.00	6.00	1,320.00	1,320.00	2,640.00	2,640.00	1,320.00	100%	1,320.00	100%
115	030716	UNDERGROUND	LS	1	1,030,000.00	1,030,000.00	1,000,000.00	1,000,000.00	1,030,000.00	1,030,000.00	1,000,000.00	1,000,000.00	(30,000.00)	-3%	(30,000.00)	-3%
116	049341	ELECTRICAL UTILITIES REMOVAL	LS	1	17,400.00	17,400.00	20,000.00	20,000.00	17,400.00	17,400.00	20,000.00	20,000.00	2,600.00	15%	2,600.00	15%
117	049342	ELEVATOR	LS	1	508,000.00	508,000.00	3,000,000.00	3,000,000.00	508,000.00	508,000.00	3,000,000.00	3,000,000.00	2,492,000.00	491%	2,492,000.00	491%
118	049343	MAINTENANCE TRAVELER	LS	1	6,640,000.00	6,640,000.00	5,000,000.00	5,000,000.00	6,640,000.00	6,640,000.00	5,000,000.00	5,000,000.00	(1,640,000.00)	-25%	(1,640,000.00)	-25%
119	049344	MAINTENANCE TRAVELER (BIKEPATH)	LS	1	370,000.00	370,000.00	2,000,000.00	2,000,000.00	370,000.00	370,000.00	2,000,000.00	2,000,000.00	1,630,000.00	441%	1,630,000.00	441%
120	049345	TRAVELER SUPPORT RAIL	KG	398,570	6.00	6.00	6.00	6.00	2,391,420.00	2,391,420.00	2,391,420.00	2,391,420.00	-	-	-	-
126	030721	NAVIGATION AND AVIATION WARNING SYSTEMS	LS	1	140,000.00	140,000.00	450,000.00	450,000.00	140,000.00	140,000.00	450,000.00	450,000.00	310,000.00	221%	310,000.00	221%
127	030722	SCADA REMOTE TERMINAL UNIT SYSTEM	LS	1	840,000.00	840,000.00	535,000.00	535,000.00	840,000.00	840,000.00	535,000.00	535,000.00	(305,000.00)	-36%	(305,000.00)	-36%
128	030723	CALL BOX SYSTEM	LS	1	453,000.00	453,000.00	100,000.00	100,000.00	453,000.00	453,000.00	100,000.00	100,000.00	(353,000.00)	-78%	(353,000.00)	-78%
129	030724	TRAFFIC OPERATING SYSTEM	LS	1	244,000.00	244,000.00	200,000.00	200,000.00	244,000.00	244,000.00	200,000.00	200,000.00	(44,000.00)	-18%	(44,000.00)	-18%
130	030725	CAMERA WITH HOUSING ENCLOSURE	EA	2	3,465.00	3,465.00	7,000.00	7,000.00	6,930.00	6,930.00	14,000.00	14,000.00	7,070.00	102%	7,070.00	102%
131	030726	PAN/TILT UNIT	EA	2	2,079.00	2,079.00	5,000.00	5,000.00	4,158.00	4,158.00	10,000.00	10,000.00	5,842.00	141%	5,842.00	141%
132	030727	CAMERA CONTROL UNIT	EA	2	3,465.00	3,465.00	5,000.00	5,000.00	6,930.00	6,930.00	10,000.00	10,000.00	3,070.00	44%	3,070.00	44%
133	030728	VIDEO TRANSMITTER DUPLEX DATA	EA	2	1,964.00	1,964.00	3,500.00	3,500.00	3,928.00	3,928.00	7,000.00	7,000.00	3,072.00	78%	3,072.00	78%
134	030729	MICROWAVE VEHICLE DETECTION SENSOR SYSTEM	EA	6	3,696.00	3,696.00	5,000.00	5,000.00	22,176.00	22,176.00	30,000.00	30,000.00	7,824.00	35%	7,824.00	35%
135	030730	FIBER OPTIC DATA MODEMS	EA	6	1,155.00	1,155.00	5,000.00	5,000.00	6,930.00	6,930.00	30,000.00	30,000.00	23,070.00	333%	23,070.00	333%
136	030731	FIBER OPTIC CABLE (72-FIBER INDOOR/OUTDOOR)	M	2,300	29.00	29.00	15.00	15.00	66,700.00	66,700.00	34,500.00	34,500.00	(32,200.00)	-48%	(32,200.00)	-48%
137	030732	FIBER OPTIC CABLE (12-FIBER INDOOR/OUTDOOR)	M	150	17.00	17.00	75.00	75.00	2,550.00	2,550.00	11,250.00	11,250.00	8,700.00	341%	8,700.00	341%
138	867130	FIBER OPTIC SPLICE CLOSURE	EA	8	1,964.00	1,964.00	1,500.00	1,500.00	15,712.00	15,712.00	12,000.00	12,000.00	(3,712.00)	-24%	(3,712.00)	-24%
139	030733	STRONG MOTION DETECTION SYSTEM	LS	1	306,000.00	306,000.00	750,000.00	750,000.00	306,000.00	306,000.00	750,000.00	750,000.00	444,000.00	145%	444,000.00	145%
140	030734	CCSF RECLAIM WATER (6 NPS)	M	640	280.00	280.00	350.00	350.00	179,200.00	179,200.00	224,000.00	224,000.00	44,800.00	25%	44,800.00	25%
141	030735	CCSF SEWER FORCE MAIN (10 NPS)	M	640	280.00	280.00	600.00	600.00	179,200.00	179,200.00	384,000.00	384,000.00	204,800.00	114%	204,800.00	114%
142	030736	CCSF WATER MAIN (12 NPS)	M	640	520.00	520.00	700.00	700.00	332,800.00	332,800.00	448,000.00	448,000.00	115,200.00	35%	115,200.00	35%
143	030737	DOMESTIC WATER (2NPS) (T1 TOWER)	M	152	449.00	449.00	400.00	400.00	68,248.00	68,248.00	60,800.00	60,800.00	(7,448.00)	-11%	(7,448.00)	-11%
144	030738	DOMESTIC WATER (2 1/2 NPS)	M	2,560	420.00	420.00	300.00	300.00	1,075,200.00	1,075,200.00	768,000.00	768,000.00	(307,200.00)	-29%	(307,200.00)	-29%
155	031195	TRANSPORTATION FOR THE ENGINEER	LS	1	1,850,000.00	1,850,000.00	4,000,000.00	4,000,000.00	1,850,000.00	1,850,000.00	4,000,000.00	4,000,000.00	2,150,000.00	116%	2,150,000.00	116%
156	031196	PHOTO SURVEY OF EXISTING FACILITIES	LS	1	15,000.00	15,000.00	100,000.00	100,000.00	15,000.00	15,000.00	100,000.00	100,000.00	85,000.00	567%	85,000.00	567%
157	031197	VIBRATION MONITORING	LS	1	15,000.00	15,000.00	280,000.00	280,000.00	15,000.00	15,000.00	280,000.00	280,000.00	265,000.00	1767%	265,000.00	1767%
158	031198	CONSTRUCTION SURVEY	LS	1	1,800,000.00	1,800,000.00	1,500,000.00	1,500,000.00	1,800,000.00	1,800,000.00	1,500,000.00	1,500,000.00	(300,000.00)	-17%	(300,000.00)	-17%
159	031199	ESTABLISH MARINE ACCESS	LS	1	35,000,000.00	35,000,000.00	35,000,000.00	35,000,000.00	35,000,000.00	35,000,000.00	35,000,000.00	35,000,000.00	-	-	-	-
166	032992	DOCUMENT MANAGEMENT SYSTEM	LS	1	1,250,000.00	1,250,000.00	1,250,000.00	1,250,000.00	1,250,000.00	1,250,000.00	1,250,000.00	1,250,000.00	-	-	-	-

04-0120F1 BID ANALYSIS 052604

ITEM NUMBER	ITEM CODE	DESCRIPTION 4/27/04 BEES	UNIT 100%	QUANTITIES 4/27/04 BEES	UNIT PRICE 4/27/04 BEES	UNIT PRICE PROBABLE BID	UNIT PRICES-BIDDER #1 INTERNATIONAL	UNIT PRICES-BIDDER #1 DOMESTIC	AMOUNT 4/27/04 BEES	AMOUNT PROBABLE BID	BID AMOUNT- BIDDER #1 INTERNATIONAL	BID AMOUNT- BIDDER #1 DOMESTIC	AMOUNT OVER (UNDER)		AMOUNT OVER (UNDER)	
										-						
Subtotal Roadway Items									**15,437,884.50**	**15,437,884.50**	**16,394,680.00**	**16,394,680.00**	956,795.50	6%	956,795.50	6%
Subtotal Superstructure Items									**11,163,580.00**	**11,163,580.00**	**13,649,220.00**	**13,649,220.00**	2,485,640.00	22%	2,485,640.00	22%
Subtotal Substructure Items									**963,200.00**	**963,200.00**	**3,612,000.00**	**3,612,000.00**	2,648,800.00	275%	2,648,800.00	275%
Subtotal Other Elements									**35,784,630.00**	**35,784,630.00**	**36,882,000.00**	**36,882,000.00**	1,097,370.00	3%	1,097,370.00	3%
Subtotal Structure Items									**47,911,410.00**	**47,911,410.00**	**54,143,220.00**	**54,143,220.00**	6,231,810.00	13%	6,231,810.00	13%
Subtotal All Items									**63,349,294.50**	**63,349,294.50**	**70,537,900.00**	**70,537,900.00**	7,188,605.50	11%	7,188,605.50	11%
Time Related Overhead-CONTRACT ITEM NUMBER 7									**#REF!**	**#REF!**	**#REF!**	**#REF!**	#REF!	#REF!	#REF!	#REF!
Subtotal									**#REF!**	**#REF!**	**#REF!**	**#REF!**	#REF!	#REF!	#REF!	#REF!
Mobilization (15%) CONTRACT ITEM NUMBER 40									**#REF!**	**#REF!**	**204,000,000.00**	**260,000,000.00**	#REF!	#REF!	#REF!	#REF!
Subtotal (AMOUNT COMPARED TO CONTRACTOR'S BID)									**#REF!**	**#REF!**	**#REF!**	**#REF!**	#REF!	#REF!	#REF!	#REF!
Cost/Square Foot									**#REF!**	**#REF!**	**#REF!**	**#REF!**	**#REF!**		**#REF!**	
Cost/Square Meter									**#REF!**	**#REF!**	**#REF!**	**#REF!**	**#REF!**		**#REF!**	
Notes:		STRUCTURAL STEEL ITEMS							#REF!	#REF!	#REF!	#REF!				
		ASSUMES INTERNATIONAL BID PRICES														

Appendix B

New Developments in Cable-Stayed Bridge Design, San Francisco

David Goodyear, Senior Vice-Pres., **John Sun,** Senior Project Eng., TY Lin International, Olympia, WA, USA

Summary

This paper describes the preliminary design process for a state-of-the-art single-tower, cable-stayed solution for the signature span of the San Francisco-Oakland Bay Bridge East Span Seismic Safety Project. This design development was carried out in the spring of 1998 as part of the seismic safety improvement for the vulnerable east spans that were damaged in the Loma Prieta earthquake in 1989. Technically progressive developments include tower shear and tension links, a dual plane splayed stay cable layout, a "double-wide" steel/concrete composite deck system utilizing precast lightweight concrete, and post-tensioned steel elements. This paper describes the design issues, design philosophy, concept development and critical details of the cable-stayed option for this new bridge.

Introduction

The preliminary design phase of the San Francisco Oakland-Bay East Signature Span included the development of both cable-stayed and suspension alternatives to a level at which these design alternatives could be compared in terms of seismic safety, aesthetics, constructibility, and project cost (*Fig. 1*). Important project constraints were imposed on the design team by the Metropolitan Transportation Commission's (MTC) Engineering Design Advisory Panel (EDAP). Principle considerations for this lifeline structure included exceptional demands for seismic performance; requirements for two separated parallel roadways; and a 161.5 m tower height limitation. These project constraints were developed in the earliest stages of the project, prior to the selection of the Design Consultant. Perhaps the most extreme criteria for design were the requirements for seismic performance. The New East Bay Bridge is a seismic safety project, owing to the severe damage experienced by the Bridge during the Loma Prieta earthquake in 1989. The proximity to the Hayward and San Andreas faults, coupled with the lifeline status of the bridge, meant that every solution had to first satisfy stringent seismic safety standards. Under these unique conditions, a wide range of structural systems were investigated, with the main variables being tower type, span layout, cable system configuration, superstructure type, and element connectivity for seismic reliability.

After extensive studies and design comparisons of the main structural elements, the Cable Stayed Alternative Design Team recommended a single-tower, cable-stayed system as shown in *Fig. 2*.

[redacted]

Paper received: May 19, 2002
Paper accepted: October 21, 2002

General Structural Configuration

The allowable tower height of 161.5 m (100 m above deck) set the practical limit of the main span length to 275 m. The design solution presented a layout with a 275 m main span and a 215 m side span. This span layout was selected as best fitting the horizontal alignment and profile that resulted from combined highway and geotechnical studies. Alternative tower shapes were investigated in the study, including tri-mast, portal and single-tower layouts. The single tower concept was the basis for the cable-stayed proposal while in pursuit of the project. The single-tower cable-stayed concept was chosen for its architectural statement, as it was clearly the most dramatic presentation when site models were developed. However, the extreme seismic criteria for the Bay Bridge site resulted in close scrutiny of system ductility under lateral load. The Engineer's commitment to the Client was to offer a single-pylon system that could achieve the same reliability as that of a traditional portal tower. This reliability was achieved with an innovative pylon link system that is described later in this paper.

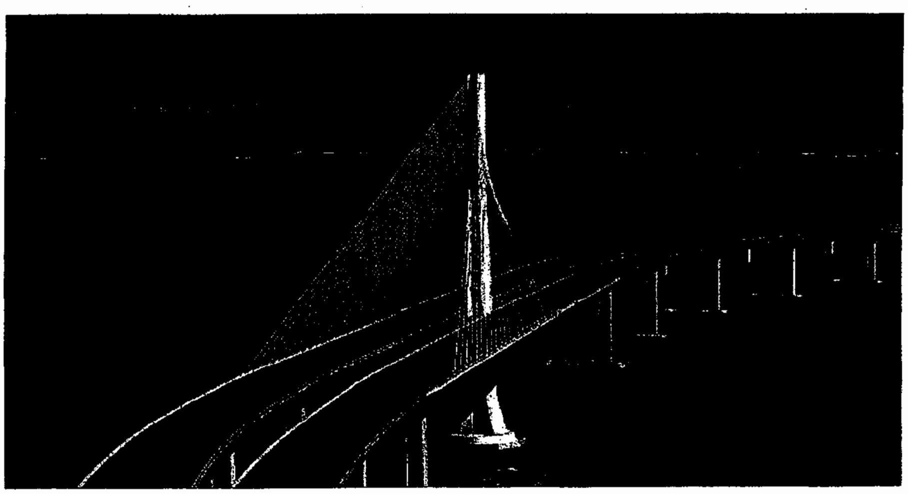


Fig. 1: Cable-stayed rendering

ance of a single tower, it is actually a tall frame. It consists of two closely spaced reinforced concrete columns that are connected by a series of steel shear links and tension links strategically placed at designated elevations of the tower. Each column is made up of a hollow, semi-elliptical cross section with an interior steel liner. The column section tapers from the tower head to the base, following the contour of the lateral force demand in the columns. The columns are connected together by shear links up to the base of the tower head, and tension links connect the columns together in the tower head to resist the tension forces from the stay cables. The links and concrete tower shafts form a transverse structural frame system with a far greater number of redundant ductile elements than a traditional portal system. The shear links are designed to be damaged in the Safety Evaluation Earthquake (SEE) event, dissipating energy of the earthquake, and thereby limiting damage to the tower shafts. The stiffness of the concrete columns and shear links are tuned such that shear link ductility demand increases progressively, while maintaining almost elastic action in the concrete elements throughout the SEE event.

This innovative structural system was conceived to improve both performance and maintainability, and to focus ductility in replaceable elements while maintaining essentially elastic response of the vertical concrete elements. The lateral displacement ductility demand was established through non-linear analysis using a special computer program, and the capacity of the tower was established by nonlinear pushover analysis. Behavior of this system is described graphically in *Fig. 4*. Note that the corresponding behavior of a conventional portal tower showed first yield of the concrete portal frame at approximately 0,5 m drift, which would require post-earthquake repair to the structure at approximate 1/3 the demand displacement for the SEE.

The shear links, as shown in *Fig. 5*, are designed as sacrificial elements to dissipate earthquake energy by yielding in shear. At the Functional Evaluation Earthquake (FEE), the links behave essentially elastically, adding significant stiffness to the towers to control displacement demands. At higher earthquake levels, including those at and above the SEE, the shear links will undergo significant shear yielding with high shear ductility demands. Significant damage to the links in the SEE will require replacement to restore the transverse stiffness and lateral load capacity of the tower. To facilitate replacement, the links are connected to the tower with bolted connections, which are designed to remain elastic for the maximum force demands expected in the link. In all cases for seismic events up to and well beyond the SEE, inelastic action and damage will be concentrated in the links and the tower concrete shafts will behave essentially elastically. Replacement of the links will be far more practical than repair to concrete sections, or to any sections that are situated over the traffic lanes. The compactness of these links eliminates the need for stiffening thin-walled steel sections, and minimizes the traditional concerns over stability and ductility in more traditional plate steel members.

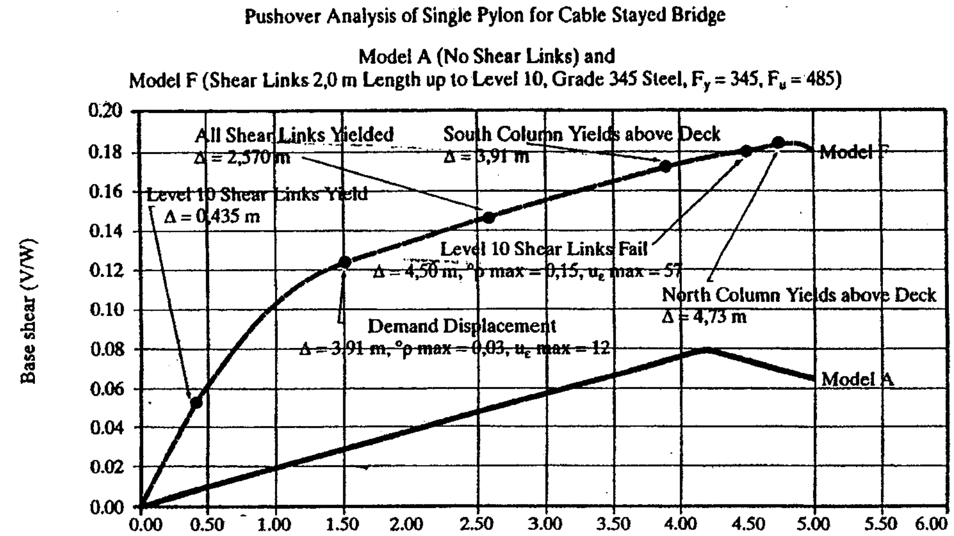


Fig. 4: Lateral load displacement ductility

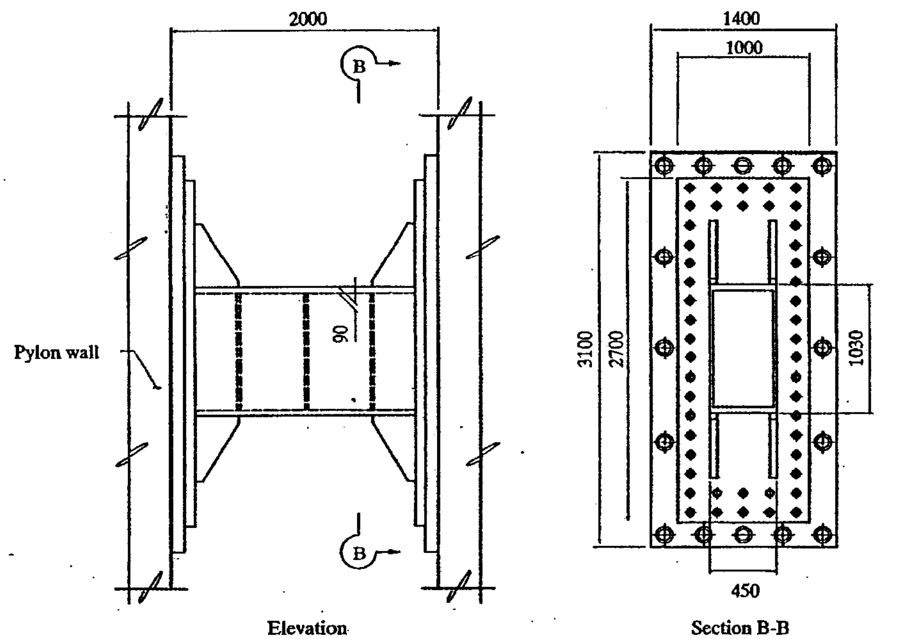


Fig. 5: Tower shear link

Due to the three-dimensional cable configuration, cable stiffness is developed in both longitudinal and transverse directions. Global transverse tension action is resisted by tension links throughout the tower head, and longitudinal splitting action is resisted by diaphragm slabs that are circumferentially prestressed. To increase the efficiency of the stay cables, the tower-head anchors for the shorter stay cables are arranged by grouping two stay anchors at one diaphragm level. This design detail also gives adequate access for stressing during construction and future maintenance.

The composite design of this tower section combines the economy of reinforced concrete tower construction with the interior steel liner and the exceptional energy dissipation capability of compact steel links, using both materials to their greatest advantage. The resulting system is a great improvement over either all-concrete or all-steel systems in terms of value, performance, and maintainability. Seismic performance exceeded that of a standard portal tower, allowing the Engineers to deliver on their commitment for seismic reliability in a single-tower design.

Cable System

The layout of stay cables is a significant element in the design of a cable-stayed bridge. In the case of the New East Bay Bridge main span, this layout carries extra significance due to the effect of high seismic demand and aesthetic sensitivity. A harp arrangement of cables was eliminated from consideration strictly on the basis of seismic response. In the case of harped cables, the lower cables could not be economically designed for the longitudinal force that developed from the axial response of the structure. A cable configuration with two-central planes was also eliminated due to the large displacement demand that developed in the tower for lateral seismic input.

A major feature of the single-tower solution is the splayed cables that run from the tower head to the sides of the double wide deck edge beams. The structural stiffness delivered by these stay cables dominates the lateral response of the tower, and lessens the displacement demand of the tower by a factor of three when compared with a tower without these transversely inclined stay cables. Wind tunnel testing of the bridge cross section also demonstrated exceptional aerodynamic performance of the proposed system. This aerodynamic stability is partially attributed to the additional torsion stiffness of the deck delivered by the inclined stay cables.

Another special issue for the East Bay Bridge main span was the extreme range of seismic force experienced by the stay cables under SEE seismic loads when compared to those under normal service loads. While the limited cycles do not place this stress range in a fatigue category, it is a design concern since there is limited information and experience of a stay cable subjected to the low cyclic, large stress amplitude loading. This concern is compounded when the minimum force constraint for a strand/wedge anchor is considered. To address the issue of stay anchor behavior, the design team recommended that specifications for the new East Bay Bridge main span include a special testing requirement for stay anchors with reference to the range of seismic response for stay cables as determined by analysis.

Double-Wide Steel/Concrete Composite Deck System

The two-plane stay cable configuration required that both inbound and outbound roadways be structurally integrated into a single deck system. With a minimum 24 m roadway width requirement to accommodate five traffic lanes and one light rail lane on both inbound and outbound roadways, as well as a 15 m roadway separation, a 70 m wide deck was designed for the stay cable supported span. This was the widest deck ever designed for a major cable-stayed bridge at this time. To reduce the seismic base shear, a light deck was a natural design preference. *Fig. 6* shows typical deck cross sections along the bridge axis. The composite superstructure system consists of a pair of longitudinal steel-concrete composite box girders. The twin girders are transversely supported by primary and secondary steel trusses alternatively spaced at 4,7 m. Both primary and secondary transverse trusses have the same dimensions. Additional transverse post tensioning tendons were added to the primary trusses to balance a portion of the dead load. The steel trusses were selected as main transverse elements of the deck system for their structural efficiency in supporting a 70 m wide deck. The concrete deck slabs are prestressed precast lightweight concrete panels interconnected with cast-in-place closure joints. This design concept of transverse trusses and the prestressed concrete panel deck reflects the modular philosophy adopted at the beginning of the project to ensure the deck quality and to enhance the deck erection efficiency.

In order to control the seismic action transferred from the deck to the pylon, a "floating" deck concept was developed with no restraints in the longitu-

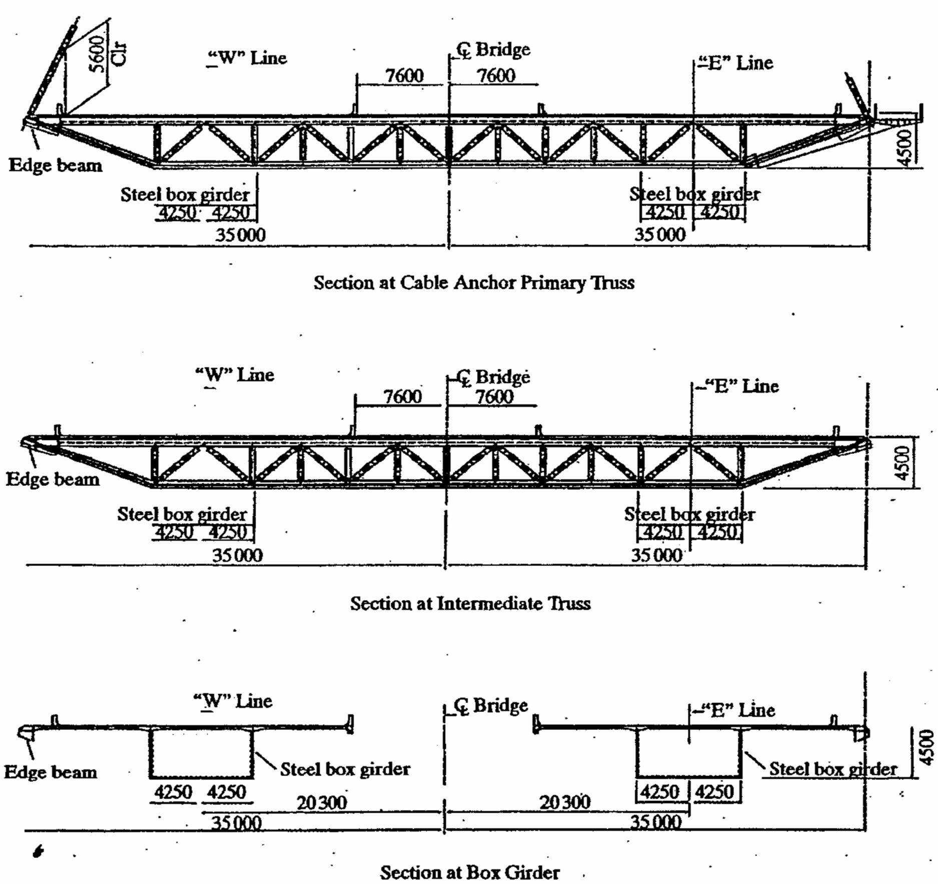


Fig. 6: Composite deck

dinal direction and limited movement connection in the transverse direction. This deck-pylon connection system made it necessary to design an adequate open space for the pylon section at deck level to allow longitudinal movement in the design seismic event. The deck-pylon intersection open space requirement results in a truncation of three transverse trusses. The solution was to add two pairs of stays on each side of the pylon supporting two pairs of primary trusses spaced at 4,7 m (instead of 9,4 m cable support in the typical deck system). The box girder flexural stiffness is adjusted to achieve a balanced loading condition between these two stays and their corresponding primary trusses.

Shear lag effects exist in any cable-stayed bridge design. Since this deck had a record-setting width of 70 m and is supported only by two planes of stays anchored at the outer edge beams, this issue was more significant in this project than any previous cable-stayed bridge. The shear lag effect would lead to a compression stress difference between the outer and inner regions of the deck. To account for this deck compression stress differential, longitudinal post-tensioning was designed near the inside edges of both roadway decks to compensate for the shear lag computed by finite element analysis of the deck system.

Conclusions

The design combination of composite deck, shear-linked tower and splayed cable configuration represents a unique and progressive solution, which is a departure from the classical design approach of a cable-stayed bridge. The innovations in this design were developed in response to the challenges of design for the unique seismic demands and architectural requirements of this bridge site. Of particular note is the excellent performance of the shear-linked pylon design, which contrasts sharply with the conventional approach of weak-column/strong beam used in seismic design of contemporary bridges. The superior performance of the weak-beam solution allows all ductility to reside in replaceable steel links, greatly improving the reliability of the vertical load carrying tower sections. The resulting structural system improves performance over traditional solutions, and provides a new benchmark in major bridge design for cable-stayed structures in regions of extremely high seismicity.

Acknowledgement

Credit goes to Dr. Brian Maroney, the Project Manager for Caltrans who was instrumental in guiding the evaluation of the link beam concept that allowed the authors to develope the single-pylon architecture for this unique site.

Appendix C

David J. Powers & Associates, Inc.
Environmental Consultants & Planners
1885 The Alameda - Suite 204
San José, California 95126
(408) 248-3500
Fax (408) 248-9641

MEMORANDUM

TO: Thomas R. Warne, Chairman
SFOBB Independent Review Team

FROM: John M. Hesler, Senior Environmental Specialist JMH

DATE: November 2, 2004

SUBJECT: **Environmental Implications Associated with Potential Changes to Design of New East Span of San Francisco-Oakland Bay Bridge**

The purpose of this memo is to generally describe the likely environmental analysis and process issues related to potential modifications to the approved design of the new East Span of the San Francisco-Oakland Bay Bridge (SFOBB). The modification being considered is a change in the design of the main span, located just east of Yerba Buena Island, from a Self Anchored Suspension (SAS) design to a Cable Stayed (CS) design. All other components of the approved project (i.e., the number of lanes, alignment, design of segments east and west of the main span, etc.) would remain unchanged.

Context for Consideration of Environmental Consequences Related to Potential Modifications

The current SAS design of the new East Span is an approved project for which extensive environmental studies have been completed. The project was the subject of a comprehensive Environmental Impact Statement (EIS) that was completed in 2001 to satisfy the requirements of the National Environmental Policy Act (NEPA).[1] Further, all required environmental permits and approvals have been obtained, including the following:

- Major Permit from Bay Conservation and Development Commission (BCDC)
- Section 404 Permit from U.S. Army Corps of Engineers
- Section 401 Certification/Discharge Permit from Regional Water Quality Control Board
- New Bridge Permit from U.S. Coast Guard
- Incidental Take Permit from California Department of Fish & Game
- Biological Opinion/Incidental Take Statement from U.S. Fish & Wildlife Service
- Biological Opinion/Incidental Take Statement from National Marine Fisheries Service
- Incidental Harassment Authorization from National Marine Fisheries Service
- Memorandum of Agreement from State Historic Preservation Officer

The fact that all of the above-described processes have been completed is important because -- as explained below -- the requirements and process for evaluating changes to an approved project are different from that for a not-yet-approved project. Specifically, FHWA's regulations for implementing NEPA state that a Supplemental EIS is not required unless changes to a proposed action would result in significant environmental impacts that were not evaluated in the original EIS. The regulations also state that FHWA can, without preparing a Supplemental EIS, approve an alternative fully evaluated in a Final EIS, even if that alternative was not identified as the preferred alternative.[2] The mechanism under NEPA for documenting the factual reasons as to why a Supplemental EIS is not warranted is a Reevaluation.

Evaluation of SAS versus CS Designs for Main Span

When considering the environmental implications of going from a SAS design to a CS design, the basic question is the following: ***How do the environmental impacts of a CS design compare to those of the approved SAS design?***

A review of the 2001 Final EIS (FEIS) provides preliminary answers to this question:

1. For the approved East Span alignment (referred to as Alternative N-6), the FEIS evaluated both the SAS and CS designs for the main span. The fact that both designs

---

[1]The project was determined to be eligible for a Statutory Exemption under the California Environmental Quality Act (CEQA).

[2]23 CFR §771.130.

were evaluated in the FEIS means that the need for future study would be less than had the CS design not been included in the document.

2. In terms of visual impacts, the FEIS concluded the following: "The main span design variations (self-anchored suspension [preferred design variation] and cable-stayed) would result in the most favorable impact upon visual quality regardless of the viewpoint location."[3] Throughout the FEIS text, there is virtually no distinction between the two designs, indicating that their visual impacts - as seen from various vantage points around the Bay - were almost identical.

3. In terms of impacts to the Bay related to the number, size, and location of piers, the FEIS does not differentiate between the CS and SAS designs. This is not surprising because I suspect that, in the context of the project as a whole, it was determined that the differences between these two designs were negligible. In other words, varying the size or number of one or two foundations/piers for these two options at the main span would not materially change the project's impact. To put these impacts in context, the FEIS stated that new fill from the entire project under Alternative N-6 was 50,400 cubic meters (66,000 cubic yards), which would result in a reduction of surface area in the Bay of 1.06 hectares (2.61 acres).[4]

4. According to the FEIS, the Coast Guard has requested a navigation channel under the main span with a minimum width of 152 meters (500 feet). Both the CS and SAS designs comply with this requirement.

5. In terms of impacts to historic resources on Yerba Buena Island, the FEIS does not differentiate between the CS and SAS designs.[5]

6. In reviewing the environmental analyses contained in the FEIS, there does not appear to be differences between the CS and SAS designs under Alternative N-6 for the following categories: air quality, noise, traffic, land use, socioeconomics, geology, hazardous waste, water quality, or cumulative.

---

[3]Section 4.3.3, FEIS.

[4]Section 4.9, FEIS.

[5]Section 4.10, FEIS.

To summarize, both the CS and SAS designs were included and analyzed in the 2001 FEIS. The analyses contained in the FEIS concluded that the environmental impacts of these two designs for the main span under Alternative N-6 were almost identical.

Conclusions and Recommendations

It is my professional judgment, based on the above analysis, that the environmental consequences of changing from a SAS design to a CS design would not be substantial. In the context of the project as a whole, this change is relatively minor and the overall environmental differences between the two designs are negligible. I recommend the following:

- Prepare a Reevaluation of the EIS
- Request Modifications to Existing Permits/Approvals, as necessary

In my experience, these tasks can easily be undertaken and completed in a period of approximately nine months. From a technical perspective, the tasks are straightforward. The biggest potential obstacle to completing these tasks in a timely manner is the workloads and priorities of the staffs of the various agencies that will be reviewing and processing the requests to amend the permits. That said, my experience on high profile projects has been that such delays have typically not materialized.

I would also point out that, depending on factors such as whether pier/foundation changes will be required to construct the CS design, it may not be necessary to modify all of the existing permits. As an example, if there will be no changes involving the piers/foundations, there would be no need to modify the Section 404 from the Army Corps of Engineers because the Corps' jurisdiction is limited to work within Waters of the United States.

Appendix D

APPENDIX D – Independent Review Team Members

The Independent Review Team is composed of professionals from all areas of the transportation and construction industry. When the original IRC was formed in September 2003 it had a membership of seven individuals. With the formation of the IRT on September 3, 2004 one IRC member, Tony Wilson has not been involved and three additional IRT members have been invited to participate. Two of these new members, John Hesler and Mike Davis, both have specific expertise in the environmental issues relating to the Bay Area and projects such as the East Span of the SFOBB. The third new team member, Peter Sanderson, has 35 years of experience in building and bidding large projects. A summary of the curricula vitae for each member of the IRC is provided below:

Thomas R. Warne, P.E. is the president and founder of Tom Warne and Associates, a management and marketing consulting firm focusing on assisting public agencies, engineering consultants and contractors in their quest for effectiveness and profitability. Mr. Warne has been involved in a number of national organizations and initiatives through much of his career. He is a past President of the American Association of State Highway and Transportation Officials (AASHTO), and spent two years as the chairman of AASHTO's Standing Committee on Highways, which is the Association's main technical body for all standards development. He continues to be involved with numerous public policy initiatives at the national level. His major project engagements include the Woodrow Wilson Bridge, the Trans Texas Corridor, Pasadena Gold Line, University Light Rail, Legacy Highway, Tri-Rail Double Track, and other major projects and programs. Prior to starting his own firm, Mr. Warne served as the Executive Director of Utah Department of Transportation (UDOT). He was appointed in 1995 by Governor Michael O. Leavitt, and for six years led Utah's third largest state agency of 1800 employees. While with UDOT he was responsible for the I-15 Reconstruction Project, which was finished 3 months ahead of schedule and $32 million under the $1.59 billion budget. The 1-15 project established design-build as the process of choice for large, high profile highway construction projects. Mr. Warne served in numerous positions with the Arizona Department of Transportation (ADOT) and as ADOT's Deputy Director and Chief Operating Officer (COO) for the last three years he was there. As the agency's COO, he was responsible for the $4.5 billion regional freeway system program in the greater Phoenix metropolitan area. Mr. Warne also served as the State Construction Engineer for ADOT where he was responsible for state's $500 million annual statewide construction program.

Tom Schmitt PE, RLS is the President of T & S Diversified, Inc. a company providing a number of services including management consulting which offers assistance with public sector administrative processes. Mr. Schmitt is a Civil Engineering graduate of Cal Poly Pomona and while in school he worked for California Department of Transportation. After graduating he became a Facility Engineer for E & J Gallo Winery, went on to be a Plant Engineer for Peter Paul Candy Company and then a representative for Garratt Callahan in Industrial Water Treatment until he joined the Arizona Department of Transportation (ADOT) as a Resident Engineer. He was later promoted to Area Engineer, Urban Highway Engineer and then District Engineer in Tucson where he was responsible for construction and maintenance for the Southwest portion of the state. Mr. Schmitt was then asked to be the Director of the Motor Vehicle Division where he was responsible to collect approximately $1 billion per year in revenue for the transportation system. In his next position as Chief Engineer for ADOT he was responsible for an annual $800 million Capital Program. Mr. Schmitt helped pass the Design Build Legislation and oversaw the first three major Design Build projects while with ADOT. After retiring from his five year tenure as Chief Engineer, he spent several years with RBF Consulting developing their Public Works section in Arizona. He has had a very diverse career and provides a valuable perspective having worked in both the public and private sectors. Over the years, Mr. Schmitt has participated in a number of local and national committees including the Standing Committee on Highways (SCOH) with Association of State Highway and Transportation Officials (AASHTO) and The Association of General Contractors of America (AGC) Transportation Committee. He is also currently the Chairman of the Friends of Civil Engineering for the Arizona State University, Civil Engineering Department as well as Chairman of the Heavy Civil Committee for the Del E. Webb School of Construction.

John R. Lamberson, a graduate of the University of California, is a member of Lamberson Consulting, a management consulting company specializing in management issues and administrative processes for construction companies. Mr. Lamberson has made the construction industry the focus of his career, specializing in insurance and bonding services to contractors. Over the last three decades, he has been involved in providing surety bond guarantees and insurance policies internationally and within the United States. In addition, Mr. Lamberson has been a member and obtained leadership positions in many construction trade associations and surety industry organizations, such as serving as Chairman of the Associated General Contractors of America’s National Associate Members Council and chairing the Affiliate and Public Awareness Committees of the Associated General Contractors of California. Other memberships include Construction Financial Management Association, The Beavers, The Moles, Building Futures Council, The Associated General Contractors of America, and The Associated General Contractors of California. He has also aided in raising funds for education in construction and often lectures and writes articles for the construction industry. In 1994, Mr. Lamberson was named winner of AGC of California’s Associate Achievement Award for many years of outstanding service to the construction industry. He was the first insurance broker ever to receive this prestigious award.

Ray McCabe P.E. is a Senior Vice President of HNTB and is the firm’s National Director of Bridges and Tunnels, which provides national oversight to the firm's bridge and tunnel design services. He is a licensed engineer in four states including California and holds a BS degree in Civil Engineering from City College of New York as well as an MS degree in Structural Engineering from Polytechnic Institute of New York.

Ray McCabe has over 25 years of professional experience, during which time he has been responsible for the structural design and/or plan production of numerous long span, movable, and complex bridge projects. Recent bridge projects for which he has played a major design role include:

- The Charles River Bridge, Boston, MA
- Storrow Drive, Boston, MA
- Goethals Bridge, Staten Island, NY
- Maysville Bridge, Maysville, KY
- Blennerhasset Bridge, Parkersburg, WV
- Bandra Worli Sea Link, Bombay, India
- Delaware Memorial Bridge, Wilmington, DE
- Cooper River Bridge, Charleston, SC
- Dames Point Bridge, Jacksonville, FL
- Maumee River Bridge, Toledo, OH
- Cape Girardeau Bridge, MO
- Many others

In addition, Mr. McCabe was member of the Constructibility Review team for the East Span Seismic Safety Project for the SFOBB in March of 2002. He has authored over 10 papers on the design and construction of long-span bridges and has received two awards from the James F. Lincoln ARC Welding Foundation for his work.

Matthew "Tim" McGowan, is a construction industry consultant with nearly 50 years of experience. Between 1957 and 1993, Mr. McGowan was employed by J.H. Pomeroy & Co., the last thirteen years of which he was its president and CEO. The company has appeared in the Engineering News Record list of the largest 400 contractors in the United States. His construction career has focused primarily on ground support systems, deep foundations, bridges, marine construction and the pre-casting of structural concrete products for major over-water bridge structures. In addition to his technical experience, Mr. McGowan has provided arbitration, mediation and dispute resolution services to the construction industry for the past 10 years. Mr. McGowan is currently a member and co-chair of the six-person California Public Works Arbitration Committee which is responsible for managing the public works arbitration system in California. He has arbitrated and mediated disputes involving intent of contract documents; disputes between owners, architects and contractors; disputes between contractors, subcontractors and material suppliers; and disputes between subcontractors. He also has experience in arbitrating and mediating disputes involving labor contracts. He is past president of the Associated General Contractors of California and of the Pile Driving Contractors Association. He is a life member of the American Society of Civil Engineers.

Terry Hays is a mechanical engineer with over 30 years of experience in engineering, design and value management for a variety of applications. He has extensive experience in leading value engineering training seminars and workshops for government, municipal and industrial clients and has participated in many detailed value engineering studies of technical facilities and processes. Mr. Hays' engineering assignments have included the design and development of components for the automobile industry, directing value engineering studies which focus on future products and development, structural analysis, and concept development for new products and projects.

Mr. Hays has conducted over 350 VE studies on a wide range of Construction projects around the world. He has served as project manager and principal team leader for indefinite quantity VE contracts with California Department of Transportation, Southwest and Pacific Divisions—Naval Facilities Engineering Command, New York City—Office of Management & Budget, and Corps of Engineers—Sacramento, Portland and Alaska Districts.

Mr. Hays has been a leader in applying the Value Engineering process to the development of program concepts (FACD) and planning strategies. Terry is experienced in conducting customer/user focus panels to identify and understand critical project issues. Results of the focus panel are directly used during the VE study. Terry has integrated focus panel and VE techniques into the Partnering Sessions, Concept Development and Planning Studies he conducts. Mr. Hays wrote the chapter on value engineering for *Maynard's Industrial Engineering Handbook* – fifth edition, published by McGraw-Hill, Inc., 2001, and he has published several papers on Value Engineering and written training manuals on value engineering that covers construction projects, product designs, manufacturing processes, and administrative systems and procedures. He is also President Elect of SAVE International.

Provisional Members

Peter F. Sanderson

Peter F. Sanderson has over 35 years of heavy construction experience both national and international in nature. The following is a brief summary of his work experience over the years:

1999-January 2004, President & CEO: Fru-Con Construction Corporation
$800 million turnover Industrial, Civil, Services, and Engineering Company.

1993-1999, President, Civil Division: Flatiron Structures Company LLC

Turnover, all in heavy civil works, grew from $50 million to $200 million.

1992-1993, President: American Bridge Company
An internationally known structural steel erector.

1981-1992, Construction Manager through Vice President: PCL Construction LTD
Worked in Canada then lead PCL's move into heavy civil construction in the USA.

1977-1981, Senior Estimator: Morrison Knudsen, Inc
Employed by MK's Northern Construction Co in Vancouver

1974-1977, Project Engineer: Theiss Brothers Pty Ltd.
Various Projects in Queensland, Australia

1972-1974, Planning Engineer: George Wimpey & Co – UK

1969-1972, Engineer: Dumez (Australia) Pty Ltd.

Education
Bachelor of Engineering, University of Western Australia-1969
Professional Engineer-British Columbia, Canada

JOHN M. HESLER
SENIOR ENVIRONMENTAL SPECIALIST/VICE PRESIDENT

Since 1982 Mr. Hesler has been Environmental Specialist/Planner for David J. Powers & Associates, Inc., San Jose, California. Prior to 1982 Mr. Hesler was Environmental Planner/Analysis for the Santa Clara County Transportation Agency/Transit District. His relevant experience has included:

- Research and prepare environmental documents required under Federal and California laws including Environmental Impact Reports, Environmental Impact Statements, Environmental Assessments, Initial Studies, Negative Declarations, Section 4(f) Evaluations, Historic Property Reports, and Army Corps of Engineers Section 404 Clean Water Act Permit Applications.

- Provide detailed analysis of potential environmental impacts of proposed private sector development and government sponsored projects, identify mitigation measures, and prepare mitigation monitoring plans. Assist in compiling appropriate findings.

- Provide experienced assistance and support in preparing planning and environmental analyses for a variety of complex transportation projects. This has included CEQA and NEPA documents for freeways, interchanges, bridges, major streets (both new construction and reconstruction), bicycle and pedestrian corridors, and airport planning. Related work has included preparation of support material such as mitigation and monitoring plans, feasibility analyses, identification of areas of impact, hazardous materials surveys, and alternatives evaluations.

- Project Manager for preparation of environmental reports on the following projects: San Jose to Gilroy CalTrain Extension, Guadalupe Corridor Serpentine/Asbestos Public Health Risk Assessment, Route 85 (West Valley Freeway), Vasona Corridor, Runway 30L Extension at San Jose International Airport, Santa Clara/Giants Stadium, Route 237 Freeway Upgrade, Yerba Buena Road/U.S. 101 Interchange, Senter Road Widening, Santa Clara County Airports Master Plan, Lawrence Expressway

HOV Lanes, Menlo Oaks Corporate Center, Saint Patrick's Seminary Master Plan, Rincon de Los Esteros Redevelopment, Watsonville Transit Ctr., Reid-Hillview Airport Tie-Downs, Cochrane Bridge and Pacheco Creek Bridge Replacements, Moffett Technology Ctr., Route 87 Freeway Upgrade, Riverpark Center, Route 17 at Lexington Reservoir Interchange, Reid-Hillview Airport Closure, San Jose International Airport Master Plan, 880/Tasman Interchange, Great Oaks Water Tank, and Metcalf Road Safety Improvements.

- Review of environmental documents from local and regional agencies for adequacy regarding transportation-related issues.

- Conducting airport and expressway noise monitoring.

- Preparation of environmental documents.